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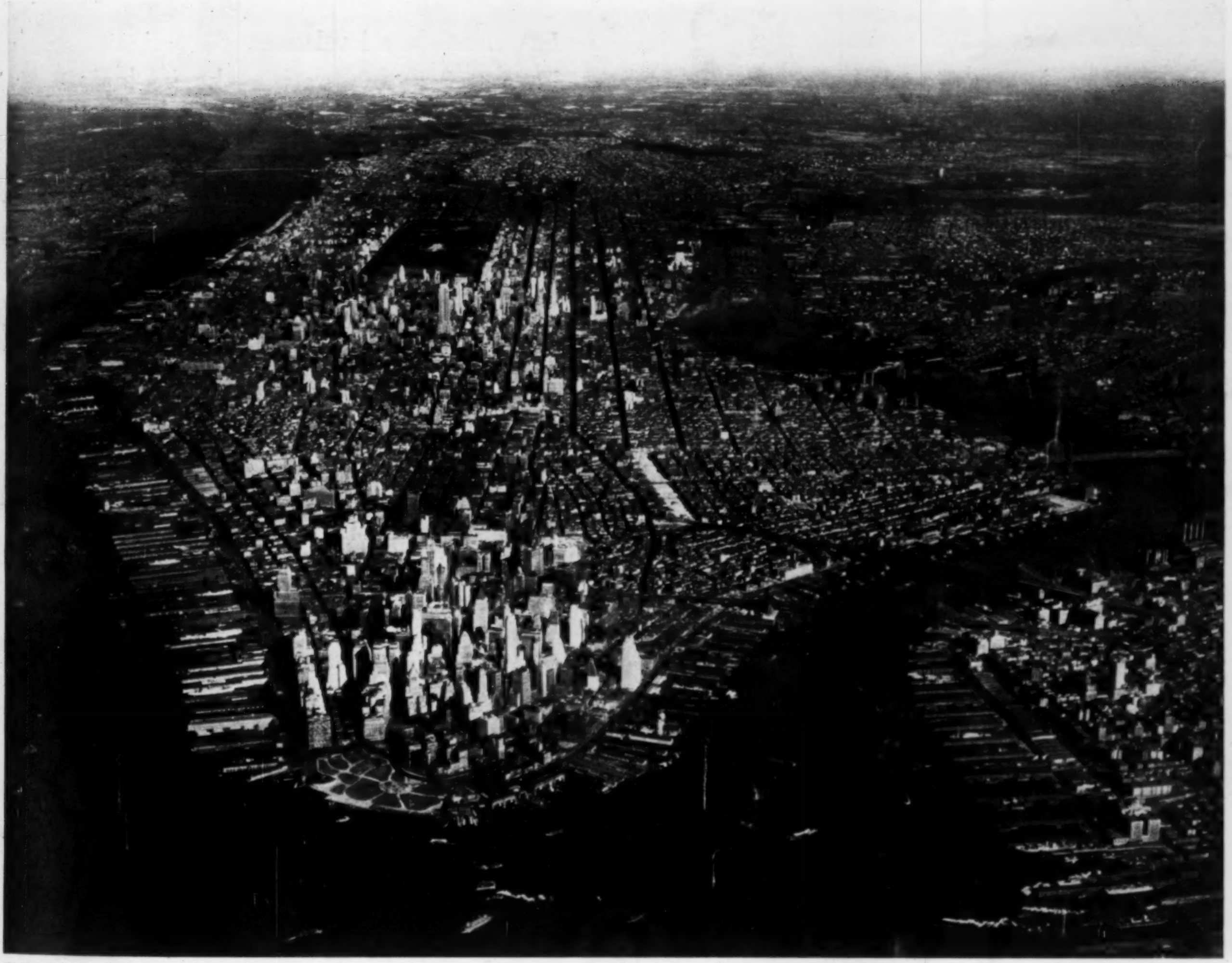

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MANHATTAN IN 1932; AVERAGE BUILDING HEIGHT SOUTH OF CENTRAL PARK IS SIX STORIES—Page 642

Volume 2 FOUNDED 1852 *Number 10*

OCTOBER 1932

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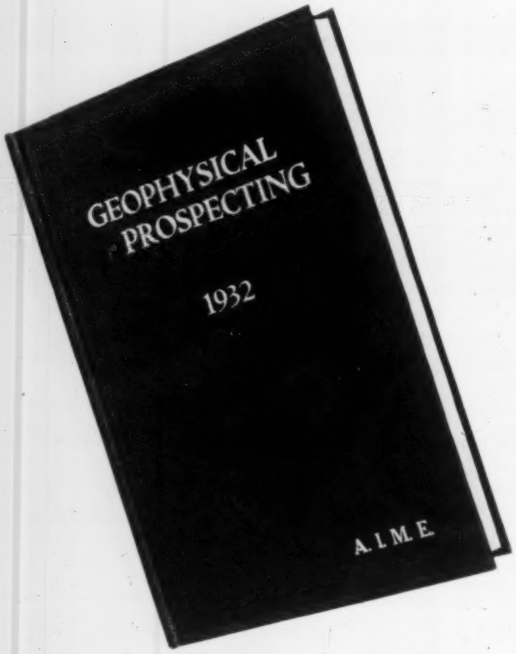

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AMERICAN SOCIETY OF CIVIL ENGINEERS

FOUNDED
1852

Abstracted from the program for public works construction presented to President Hoover on May 19, 1932

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A. I. **Raisman** is a graduate of the School of Mines of Columbia University. He has been in the employ of the City of New York in transportation work for more than thirty years, beginning as Structural Draftsman on the design of the first subway, and passing through the various grades of Assistant Engineer, Division Engineer, and Chief Designing Engineer on the various rapid transit systems built by the city.

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J. S. **Worley** has been engaged since 1900 in the location and construction of steam, interurban, and street railways in the Middle West, in superintendence of construction of many important public works, and in appraisal of many public properties. Between 1913 and 1920 he was a member of the Engineering Board of the Interstate Commerce Commission, for which he was Consulting Valuation Engineer in 1921. In the following year he became Professor of Transportation at the University of Michigan.

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Sherwin F. Kelly became interested in geophysics while holding a two-year graduate fellowship for study in French universities. Later he taught geology and mineralogy at the University of Toronto and was connected with field surveys in the United States, Canada, Algeria, Morocco, and France, utilizing geophysical methods. Since 1930 he has lectured on geophysics at various universities. He maintains a consulting office in New York in partnership with Bela Low.

J. E. **Willoughby**'s entire professional work has been in the location, construction, and maintenance of railways and appurtenant facilities. His work has included responsible charge of heavy mountain construction, such as that on the Knoxville, Lafollette and Jellico and the Lexington and Eastern railroads, through the mountains of eastern Tennessee and Kentucky; revision of line and grade on the Kentucky and Tennessee main lines; and lighter work on second track construction and new lines on the Atlantic Coast Line system.

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Thomas Adams has prepared plans or acted as consultant to over a hundred American, Canadian, and English cities and towns. He was General Director of the Regional Plan of New York and Its Environs during the preparation of the plan and more recently acted as consultant to the continuing Regional Plan Association. He still maintains an active consulting practice through the London firm of Adams, Thompson, and Fry.

Volume 2 Number 10

October 1932

Entered as second class matter September 23, 1930, at the Post Office at Easton, Pa., under the Act of August 24, 1912.

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Printed in the U. S. A.

CIVIL ENGINEERING

Published Monthly by the

American Society of Civil Engineers
(Founded November 5, 1852)

Publication Office: 20th and Northampton Streets, Easton, Pa.
Editorial and Advertising Departments:
33 West 39th Street, New York

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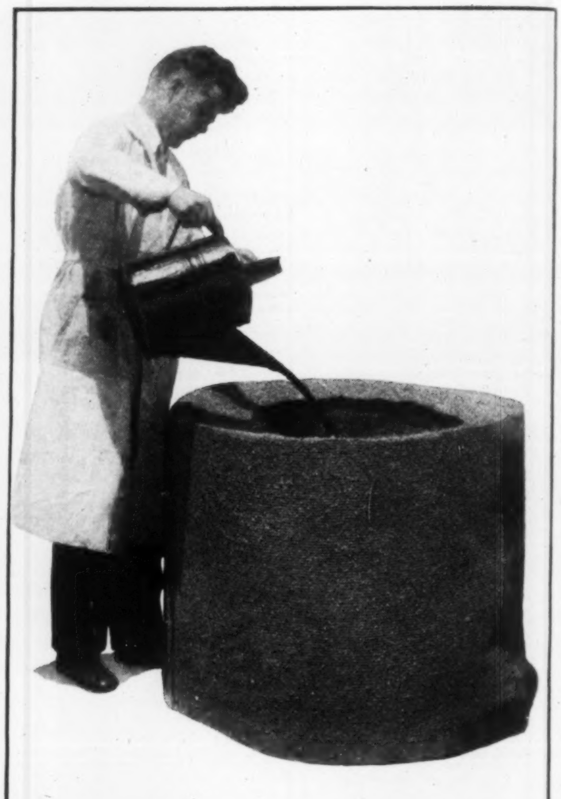

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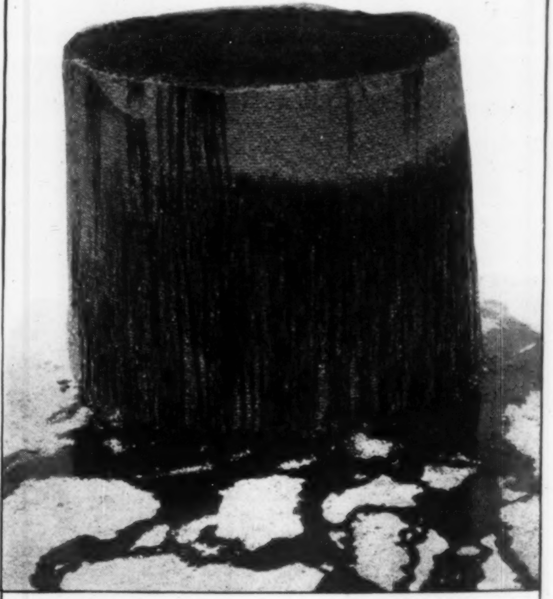

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OCTOBER 1932

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VOLUME 2

NUMBER 10

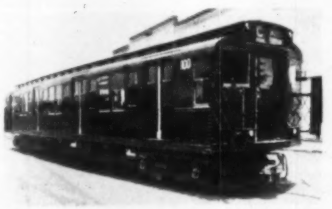
Evolution of the Subway Station

Complex Problems Progressively Solved in the Expansion of New York's Vast Underground System

By A. I. RAISMAN

MEMBER AMERICAN SOCIETY OF CIVIL ENGINEERS
CHIEF DESIGNING ENGINEER, BOARD OF TRANSPORTATION, CITY OF NEW YORK

TO most people, the New York subway systems are a maze—a curious and wonderful evolution, misunderstood but efficient, uncomfortable but useful. They are concerned not with the means adopted but with the results obtained, leaving it to the engineers to evolve, somehow, a workable system. One of the most difficult problems, often unappreciated even by engineers, is the station itself, its requirements and its physical limitations, as described in this article. After years of study and experience, Mr. Raisman relates here in an interesting manner some of the problems that have been met and the conclusions reached. This paper, especially timely because of the opening on September 10 of the new Eighth Avenue line, is an abstract of that read November 18, 1931, before the Metropolitan Section.



PASSENGER CAR FOR EIGHTH AVENUE SUBWAY

CONSTRUCTION on the subways of New York began in 1900 when the Interborough Rapid Transit Company (I. R. T.), as the assignee under contract with the city, commenced the line extending from the intersection of Park Row and Broadway, near the City Hall, along Park Row, Lafayette Street, and Fourth Avenue to the Grand Central Terminal, and thence west on 42d Street to Times Square and up Broadway to two termini, one in the Bronx and the other at Van Cortlandt Park. From the City Hall, the subway was extended to the southern tip of Manhattan at the Battery, and under the East River to the business section of Brooklyn. In 1913 the city entered into a contract with the I. R. T. and one with the Brooklyn Rapid Transit Company (now the Brooklyn Manhattan Transit Company, or B. M. T.), the two together being known as the Dual Contract, which resulted in the completion of both an east side and a west side line in Manhattan as well as an intermediate line under Broadway, south of 59th Street, and numerous extensions in the boroughs of Brooklyn, the Bronx, and Queens. These separate systems, having a total route length of more than 114 miles, were constructed at a final cost of about $400,000,000, exclusive of equipment. The cost was borne in part by the city and in part by the operating companies under certain provisions by which the ownership will revert entirely to the city at the expiration of the operating franchises.

In 1925 the City of New York began the construction of an independent, city-owned system, to comprise 55 miles of route in the boroughs of Manhattan, the Bronx, Queens, and Brooklyn. The first unit of this system, that along Eighth Avenue in Manhattan, has just been put in operation under city management on September 10, 1932. The cost of construction and real estate will total about $550,000,000 and that of equipment, another $135,000,000. Thus New York's investment in these subway systems exceeds one billion dollars for construction, equipment, and necessary real estate.

SUBWAY STATIONS ESPECIALLY SHOW IMPROVEMENT

Early subways in New York were somewhat experimental in nature, for there were few precedents to follow. Everything considered, they stand today as a remarkable tribute to the foresight of the pioneer engineers on this work, especially the chief engineer, the late William Barclay Parsons, Hon. M. Am. Soc. C.E. In the main they are admirably serving their purpose in the face of demands far beyond the conception of their designers. Nevertheless, all phases of subway planning have been subjected to much study and as a result have been greatly improved with the passing of years. No one element shows this more clearly than the subway station itself. This structure is the first, if not the main object noticed by the passenger. It is affected by an endless variety of limiting or desired conditions. It is therefore not only one of the most vital, but also one of the most interesting parts of subway work from the standpoint of the engineer.

It is axiomatic that no two subway stations are exactly alike. No attempt will therefore be made to cover more than a few of the many considerations governing the major features of the station design. The features discussed will be illustrated by reference to the subway systems of greater New York.

LONGER STATIONS REQUIRED

In the course of time, a material change has been made in the length and spacing of stations. The drawings for the first subways, now operated by the I. R. T., show local and express stations 200 ft. long (Fig. 1). During construction it was decided to make the express stations

350 ft. long for trains of eight 50-ft. cars but to leave the local stations 200 ft. long to accommodate five-car trains. After intensive operation began, the length for express stops was increased to 435 ft., or to 480 ft., to provide for ten 51-ft. cars.

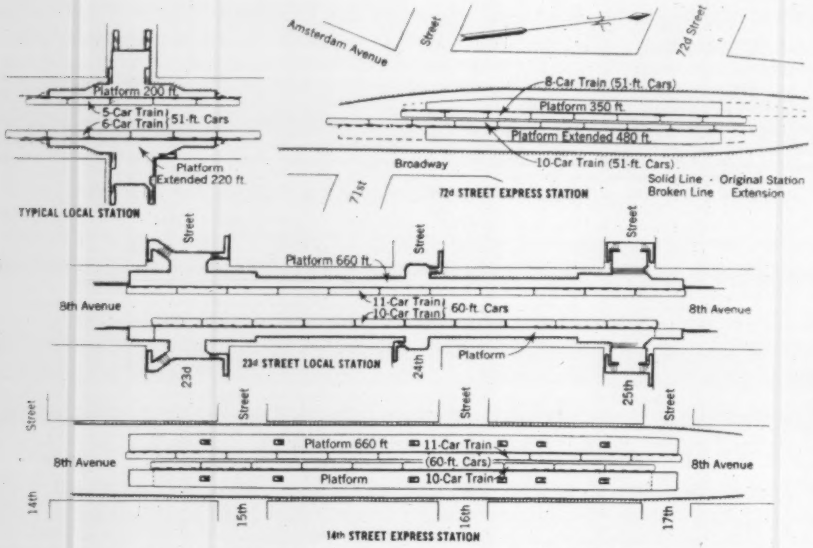

Platform 200 ft.
5-Car Train
6-Car Train
51-ft. Cars
Platform Extended 220 ft.
TYPICAL LOCAL STATION
Amsterdam Avenue
Street
72d Street
8-Car Train (51-ft. Cars)
Platform 350 ft.
Platform Extended 480 ft.
10-Car Train (51-ft. Cars)
Broadway
71st
72d STREET EXPRESS STATION
Solid Line - Original Station
Broken Line Extension
Street
Platform 660 ft.
11-Car Train
10-Car Train
60-ft. Cars
8th Avenue
8th Avenue
Platform
23d
24th
25th
23d STREET LOCAL STATION
Street
Platform 660 ft
11-Car Train
(60-ft. Cars)
10-Car Train
Platform
8th Avenue
8th Avenue
14th
15th
16th
17th
14th STREET EXPRESS STATION

FIG. 1. LENGTH OF PLATFORMS IN SUBWAY STATIONS
Above: As Planned in 1900 for Original Construction on the Interborough Rapid Transit Line
Below: Local and Express Stations on the Independent City-Owned System

Where the platform is 435 ft. long, the center and rear doors of the last car and the center and front doors of the first car of these ten-car trains cannot be opened at the station, whereas if it is 480 ft. long the center doors of these cars are usable. When, as at present, two trainmen control all the doors of a ten-car train, the inner-end and center doors of end cars are opened together, except at platforms 435 ft. long.

Subsequently, the length of local stations was also increased, to about 225 ft., to permit the operation of six-car trains.

The standard length of platforms on the lines of the B. M. T. is now 530 ft. to provide for the operation of trains made up of eight 67-ft. cars. On the new independent system, both local and express platforms are 600 ft. long to accommodate trains of ten 60-ft. cars, and provision has been made for lengthening them to 660 ft. to take eleven 60-ft. cars, as shown in Fig. 1. Where conditions warrant, platforms are made the full 660-ft. length initially.

With the extension of the length of platforms to more than two blocks has come the general policy of locating passenger controls at each end of the station instead of at its center. This has permitted an increase in the distance between stations. The aim has been to place stations one-half mile apart on centers (about ten longitudinal blocks in Manhattan), but special conditions have made it necessary to vary this distance. Generally, the number of passenger entrances has been increased over that provided in former designs with the result that the maximum longitudinal walking distance to a station entrance has been reduced to four blocks.

The average distance between station centers is 0.33 mile on the I. R. T. Broadway-Seventh Avenue Subway (Manhattan) between the Chambers Street and 59th Street stations, and 0.47 mile for the parallel Eighth Avenue Subway of the independent line between these same limits. Reducing the number of stations and increasing the distances between them means increased speed of operation. By placing the passenger controls at the ends of the stations, the length of walk required is not increased. Allowing for a stop of 20 sec., plus time for deceleration and acceleration of trains and for signaling, each station stop requires about one minute more than the time required for constant speed. One minute therefore represents the loss of time due to a station stop. In addition, the greater distance between stations enables trains to accelerate to maximum speed, which is not always possible with the closer spacing.

EXPRESS STOPS IN CONGESTED DISTRICTS

Advances have also been made in the arrangement of the express stations on the new city-owned line. A long stop at stations seriously interferes with the intensive operation of rapid transit lines. At the Times Square station of the I. R. T. the stop has sometimes been as long as 2 min., although the usual length of stop there is 60 sec. By reducing this time, more trains can be operated past a given point in a given interval of time.

One of the chief causes of prolonged station stops is the transfer of passengers between express and local trains, especially in congested business districts. On the new Eighth Avenue system, all but three of the stops in the congested district between Chambers and 59th streets are express stations, where all trains will stop. Thus there is no incentive for passengers to change from a local to an express train at the entrance to that district.

In the congested business district between Chambers Street and 59th Street on the Broadway-Seventh Avenue (I. R. T.) Subway, there are eight local stations at which the expresses do not stop, as compared with three on the newer Eighth Avenue Subway. In addition, the last-

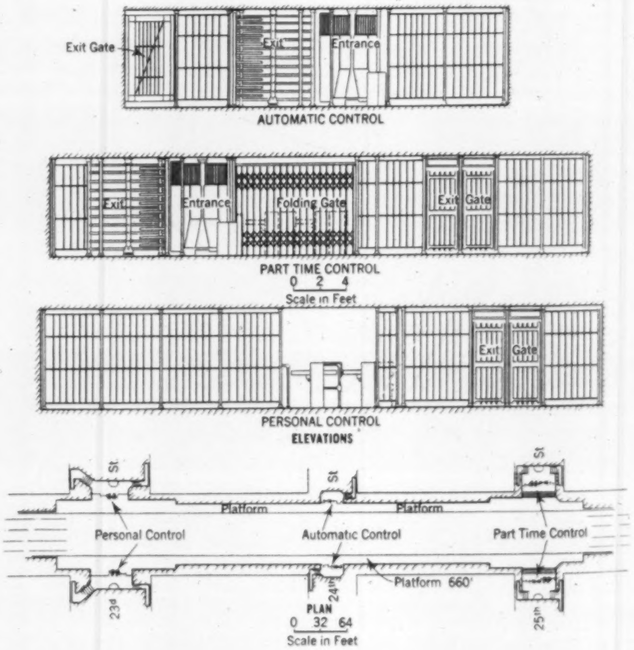

Exit Gate
Exit
Entrance
AUTOMATIC CONTROL
Exit
Entrance
Folding Gate
Exit Gate
PART TIME CONTROL
0 2 4
Scale in Feet
Exit Gate
PERSONAL CONTROL
ELEVATIONS
St
St
St
Platform
Platform
Personal Control
Automatic Control
Part Time Control
Platform 660'
23d
24th
25th
PLAN
0 32 64
Scale in Feet

FIG. 2. TYPICAL ARRANGEMENT OF PASSENGER CONTROLS
At the 23d Street Local Station of the New Eighth Avenue Subway

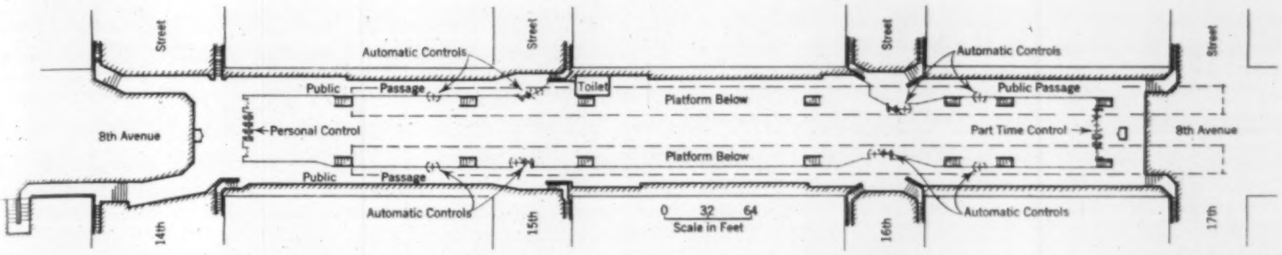


Fig. 3. Arrangement of a Station with Mezzanine Access
14th Street Express Stop on the New Eighth Avenue Subway

mentioned line has a long express run from 59th to 125th Street. In that stretch the express trains will be able to make quick time and there will also be the advantage that the local trains will pick up heavy loads. For southbound traffic originating along Eighth Avenue south of 125th Street, there will be practically no incentive to transfer from a local to an express train. As a result the locals will doubtless handle a large part of the through traffic of that section, and all four tracks will be taking heavy loads. Where the locals act only as distributors or collectors for the express trains, the local tracks are operated only at partial capacity.

On the new Eighth Avenue line the typical side platforms of the local stations are usually about 18 ft. below the street level and have controls at each end (Fig. 2), or four in all. This means heavy operating expense if money-changing agents are stationed at each control. To reduce this expense, mechanical controls are to be used at points of light traffic. The ordinary type of low turnstile and railing is provided where there are change agents, and the high automatic turnstiles with high railings are used where there are no agents.

Both high and low turnstiles may be provided at certain points where it is expected that a change agent will be in attendance during rush hours. At other times passengers can pass through the control if they have the proper coin. Otherwise it will be necessary for them to go to the personal control entrance. Automatic change machines may be installed later to remove this difficulty.

MEZZANINE A USEFUL FACILITY

At express stations, with island platforms, access is usually gained through mezzanines above the platforms. Mezzanines can usually be provided if the distance from street to platform level is more than 24 ft. This type of station has the advantage that the number of controls can be reduced to two, one at each end, and either platform can be reached without crossing the street at the surface. A typical layout for a mezzanine station is shown in Fig. 3. Personal control may be used at one end and mechanical control at the other. However, in such a case a passageway is generally built outside the control area, so that a passenger can walk to the other end of the station without climbing to the street and descending. A further advantage of this passageway is that at any busy locality, such as is shown in Fig. 3, additional entrance stairways can be provided at cross streets or through adjacent buildings.

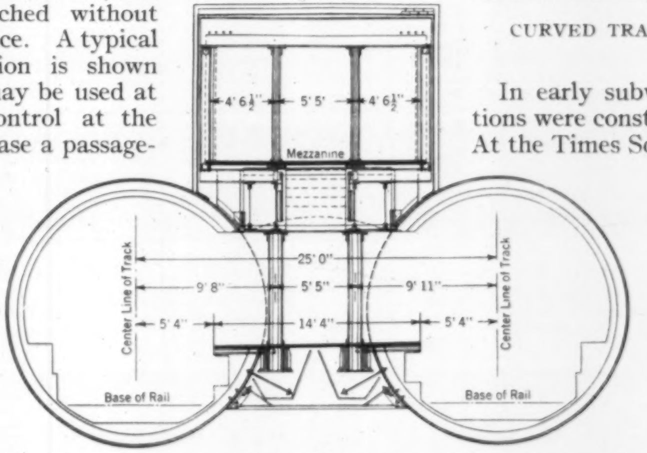


Fig. 4. An Island Station on Willoughby Street, Brooklyn
Narrow Station Platform and Mezzanine of the B. M. T. System

Passengers can then walk underground to the personal control, at whichever end it may be, or mechanical control may be installed opposite such stairways.

A similar arrangement can be used at local stations with side platforms where the subway is deep enough for a mezzanine. Here a free longitudinal passageway is provided along the center line of the mezzanine so as not to interfere with the stairways from the mezzanine to the platform. Such an arrangement is inferior to the island platform type in that it does not permit of entrances from cross streets or buildings—a disadvantage sometimes overcome by cutting across the control area. However, it is economically superior because it requires only one change agent. Also it permits both north and southbound passengers to enter the subway on either side of the street.

Where the structure must be deep for other reasons, a continuous mezzanine over the tracks and platforms has been provided at but little additional expense. The question has been raised as to whether a large area in the mezzanine is objectionable because of maintenance expense. Should it prove uneconomical to keep this entire space for station purposes, certain parts could be set aside for storage. At important points it is proposed to construct stores in the mezzanines and rent them to concessionaires, thus providing the city with a considerable income.

At points where the highway traffic is heavy, as on Queens Boulevard, Borough of Queens, practically all the stations have been depressed sufficiently to accommodate a mezzanine, so that passengers can enter or leave the subway on the most convenient side and avoid crossing the highway above. An important advantage of the continuous mezzanine is that passengers need not walk along a congested platform in going to and from the street, but can use the larger open space on the mezzanine. This consideration has permitted very narrow station platforms at certain stations.

CURVED TRACKS COMPLICATE STATION STRUCTURE

In early subway work, a number of stations were constructed on very sharp curves. At the Times Square I. R. T. station, where the inner track had a minimum radius of about 410 ft., the projection of the ends of the cars beyond the track made it necessary to set back the platform edge so that there were spaces of 10 or 12 in. between the center doors of cars and the platform. Many suits were brought against the operating company by passengers who claimed they had received injuries as a result of stepping into these open spaces or being pushed into them by the dense crowds. The curved track at this station has now been abandoned, except for transfer shuttle service. At the City Hall loop (I. R. T.) station, however, there is still a track with a radius of 147.25 ft., forming an opening between the center doors of cars and platform edge of as much as 28 in. At this station, in order to avoid this gap, only the end doors of cars are opened. At some curved stations, movable grids in the platform opposite the car doors are pushed out mechanically to fill the gaps when the train stops and withdrawn as it starts.

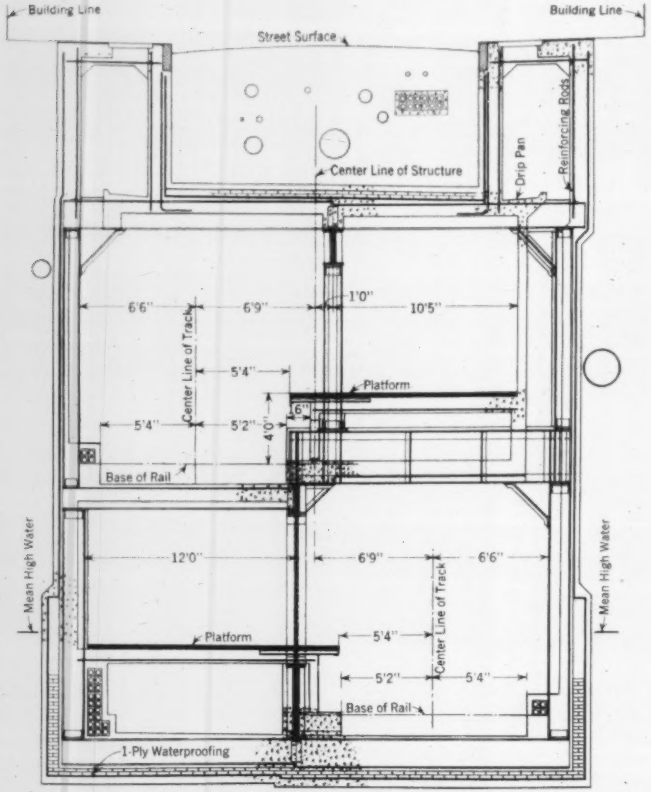

Building Line
Building Line
Street Surface
Center Line of Structure
Drip Pan
Reinforcing Rods
6'6"
6'9"
1'0"
10'5"
Center Line of Track
5'4"
Platform
6"
5'4"
5'2"
4'0"
Base of Rail
Mean High Water
12'0"
6'9"
6'6"
Center Line of Track
Mean High Water
Platform
5'4"
5'2"
5'4"
Base of Rail
1-Ply Waterproofing

FIG. 5. A TWO-STORY STATION IN A NARROW STREET
Fulton Street Station on the Nassau Street Line of the B. M. T. System

As a result of these experiences with what one of my friends calls "watch-your-step-stations," all platforms are now constructed on tangents or on a radius of 3,000 ft. or more. With such a radius, the platform edge needs to be set back a maximum of only about 2 in. more than normal. This amount is not considered objectionable. In a few instances it has been necessary to use a radius of about 2,000 ft.

SPECIAL PROBLEMS SOLVED

Some of the problems met with in station design can best be illustrated by specific structures, such as the Willoughby Street station in Brooklyn (Fig. 4) on the B. M. T. line, which was built after the subway was in operation. The trainways were in cast-iron tubes spaced 25 ft. apart on centers, so that the island platform between tracks could be only about 15 ft. wide; and for convenience of construction, columns were placed alongside the tubes about $5^1/_2$ ft. apart on centers laterally. The result was a narrow platform cluttered with columns; but a sufficient number of narrow stairways were provided between the rows of columns from the platform to the mezzanine so as to make practically unnecessary any longitudinal movement of passengers on the platform.

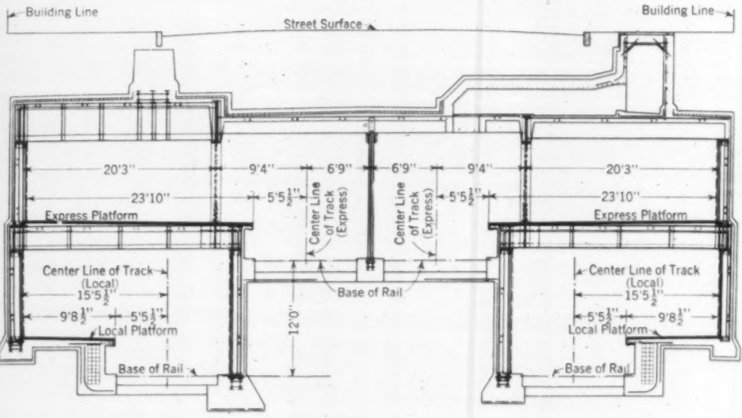

Building Line
Building Line
Street Surface
20'3"
9'4"
6'9"
6'9"
9'4"
20'3"
23'10"
5'5½"
Center Line of Track (Express)
Center Line of Track (Express)
5'5½"
23'10"
Express Platform
Express Platform
Center Line of Track (Local)
15'5½"
9'8½"
5'5½"
Local Platform
Base of Rail
12'0"
Base of Rail
Center Line of Track (Local)
15'5½"
5'5½"
9'8½"
Local Platform
Base of Rail

FIG. 6. DEPRESSED LOCAL TRACKS PROVIDE WIDE EXPRESS PLATFORMS
Nostrand Avenue Station on the Fulton Street Line of the Independent, City-Owned System

Along the B. M. T. Subway in Nassau Street, Manhattan, another solution was found. The broken street lines required curves in the tracks, so that the maximum feasible width of structures in the vicinity of Fulton Street was about 31 ft., permitting an island platform less than 8 ft. wide. As two-way traffic was to be expected at this station, the minimum width desirable for such a platform was 20 ft. A double-deck arrangement was therefore chosen, as shown in Fig. 5, with the upper track over the lower platform and the lower track under the upper platform.

At Beekman Street it was necessary for the Nassau Street tracks of the B. M. T. to cross over the I. R. T. Subway, a requirement that made it impossible to use at this station a layout with track above track and platform above platform. There was not sufficient room longitudinally for the northbound track to drop down to the second level and then swing over to the west before reaching the station. Therefore a left-handed entrance to the station was required. The southbound track is on the upper level and its platform is on the east, instead of the west side. Similarly, the northbound track, on the low level, has its platform on the west side of the street, in reverse of the usual position.

ROOM GAINED BY DEPRESSING LOCAL TRACKS

An interesting solution was found for the problem at the Nostrand Avenue express station of the independent, city-owned line, on Fulton Street in Brooklyn. Since Fulton Street is only 80 ft. wide, the maximum width available for each island platform was only about 12 ft. if all four tracks were on one level. The structure at Nostrand Avenue might have been extended into adjacent property in order to obtain platforms of the desired 20-ft. width except that the right-of-way costs would have been very high, probably $2,700,000. Another possible solution was to make a double-deck station, but that would have required difficult flexing and curving of tracks, and the lower base of rail would have had to be 37 ft. below the street. It was desirable to keep the structure as near the surface as possible to facilitate a future undercrossing for a line on Nostrand Avenue.

In the ordinary mezzanine station in which all tracks are on one level, the base of rail must be about 30 ft. below the street

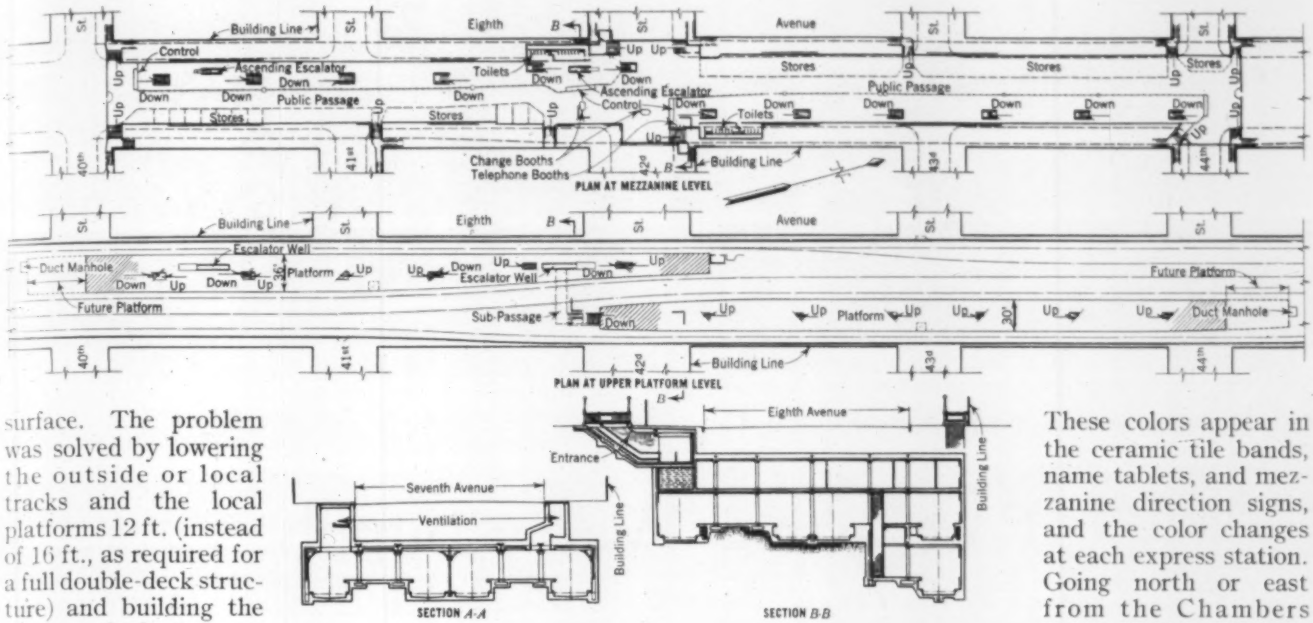


Fig. 7. Tandem Arrangement of Platforms to Increase Their Width

The 42d Street Station on the New Eighth Avenue Subway
Section *A-A* Is through the Times Square Station of the
I. R. T. Seventh Ave. Subway

surface. The problem was solved by lowering the outside or local tracks and the local platforms 12 ft. (instead of 16 ft., as required for a full double-deck structure) and building the express platforms over the local tracks and platforms, as shown in Fig. 6. Thus the low-level tracks are 33 ft. below the street surface; the tracks are not curved; wide platforms are provided for the express trains; and the entire station is within the street lines, the total width of the structure being about 74 ft. The cost of construction was slightly less than that for the other designs, and the cost of taking private property was entirely avoided.

WIDENING FOR EXPRESS PLATFORMS

On the old east-side I. R. T. Subway, the width of the island platforms at the former Grand Central station was 20 ft. and that of the main sections of the Brooklyn Bridge station was also 20 ft. At the Times Square station, on the Broadway-Seventh Avenue I. R. T. Subway, built later, the platform width was increased to 22 ft., the maximum that could be used in this 100-ft. street after leaving the minimum allowable space for sewers on each side. Since the traffic at this station is tremendous, additional width of platform would have been very advantageous, but it was not feasible to provide it. Just west of Times Square, on the new Eighth Avenue Subway, the platforms at 42d Street have been increased to 30 and 36 ft., respectively, for the east and west sides, by means of the tandem arrangement shown in Fig. 7.

The northbound platform, extending from 42d to 44th streets, was made 30 ft. wide, and the southbound platform, running from 40th to 42d streets, was made 36 ft. wide. Underneath the southbound platform a low-level platform was built in order that trains from Queens can operate on the tracks of the Eighth Avenue Subway south of 42d Street without interfering with the southbound traffic entering this station from Washington Heights. The extra width of the upper southbound platform was provided because of the reduction in its effective width caused by the escalators and stairways extending through it from the low level to the mezzanine.

In the city-owned line a number of new features are being introduced. One of these is to make stations more quickly identifiable. A system of five colors—violet, blue, green, yellow, and red—has been used in regular rotation in the ornamentation of successive stations. These colors appear in the ceramic tile bands, name tablets, and mezzanine direction signs, and the color changes at each express station. Going north or east from the Chambers Street-Hudson Terminal station, the local stations take the color of the express station immediately preceding. The color bands in the express stations are wider than those in the local stations.

In the first series of five colors, two shades of the color are used in each station. In the second series, each color is used in combination with black. To further aid in the quick identification of stations, interchangeable unit tile plaques indicating the station name or street are inserted along the side walls under the color bands, and spaced from 5 to 10 ft. apart on centers. The wall finish of stations may be of tile or enameled brick. Spaces for advertising posters are provided along the walls of the mezzanines in express stations and on the side walls of the local stations. These spaces are slightly recessed, cement-finished panels for standard-sized posters. Instead of being plastered, the ceilings have a smooth or rubbed surface of concrete ready for painting.

Entrances from malls, isles-of-safety, or small isolated blocks or parks are in general avoided, as street traffic makes them difficult of access. At some stations, where the width of the platform is limited by narrow streets, platform space is saved by omitting a line of columns.

The change booths used are metal, with standard fronts. Wherever possible they are placed in the open and provided with windows on all sides, to give the agents a greater range of vision. Passimeters are of the mechanical, noiseless type.

Obviously, in such a short paper only a few of the more spectacular problems of station design that have confronted engineers working on New York's subways can be mentioned. Enough has been included, I hope, to throw light on the complexity of the factors entering into the solutions. The principles enumerated should be broad enough in application to serve engineers elsewhere who may be called upon to deal with similar problems.

The Independent System is being constructed by the Board of Transportation of the City of New York, of which the Hon. John H Delaney is Chairman, and the Hon. Daniel L. Ryan and the Hon. Frank X. Sullivan are Commissioners. Mr. Robert Ridgway, Past-President of the Society, is Chief Engineer, and the late John R. Slattery, M. Am. Soc. C.E., was Deputy Chief Engineer.

SAMPLING THE SNOW COVER

WEIGHING THE SNOW SAMPLE

Forecasting Water Supply

Snow Surveys in Utah Predict Run-Off from Four to Eight Weeks in Advance

By GEORGE D. CLYDE
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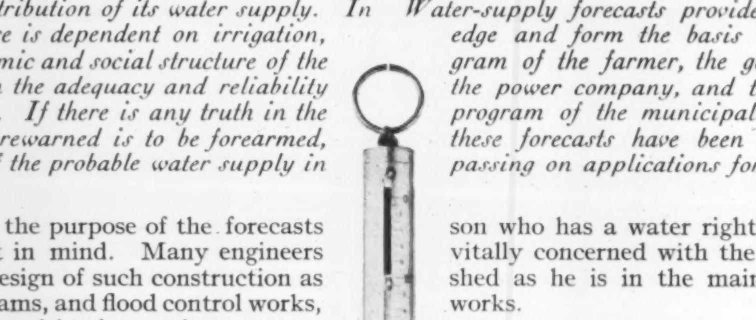
IRRIGATION ENGINEER, UTAH AGRICULTURAL EXPERIMENT STATION, LOGAN, UTAH

DEVELOPMENT and growth of a community in an arid or semi-arid region are measured by the amount and distribution of its water supply. In the West, agriculture is dependent on irrigation, and the entire economic and social structure of the people is built upon the adequacy and reliability of the water supply. If there is any truth in the saying that to be forewarned is to be forearmed, then a knowledge of the probable water supply in advance of its occurrence is of great value in making the most complete utilization of the water when it comes. Water-supply forecasts provide this advance knowledge and form the basis of the planting program of the farmer, the generating program of the power company, and the water-conservation program of the municipality. In many cases these forecasts have been used by bankers in passing on applications for loans.



THE UTAH SNOW SCALE AND SAMPLER

IN forecasting run-off, the purpose of the forecasts must always be kept in mind. Many engineers are engaged in the design of such construction as culverts, storm drains, dams, and flood control works, and are therefore interested in the maximum storm or flood flow that will occur. They want information on the time of run-off, its concentration, and the maximum rate of flow rather than on its total amount. On the other hand, where the water is to be used or consumed, the question of total amount as well as that of seasonal distribution is important.

Streams that furnish water for irrigation are annually replenished from the accumulated snows on the high watersheds. The areas that contribute water to the streams, either over, on, or under the surface of the ground, are known as the watersheds of the stream to which they contribute. In the past, the irrigator considered his water supply only at the point of diversion from the river into the irrigation canal. He was not interested in the source of the water; he only wished to know how much water was in the river at the point of diversion. The continued increase in water consumption and decrease in water supply during the past few years have compelled the focusing of attention on the source of the supply. The time has now come when every person who has a water right in any stream is just as vitally concerned with the conditions on his watershed as he is in the maintenance of his irrigation works.

SNOW ACCUMULATIONS MEASURED

In an arid climate the water supply is derived largely from precipitation on high mountains adjacent to valleys. Where this precipitation for the most part falls as snow, the accumulated precipitation may be measured in advance of its appearance as run-off. From such measurement and a consideration of other factors, it is possible to predict the run-off from 30 to 60 days in advance of its occurrence.

In the West there are two distinct seasons, one wet and one dry. It so happens that the growing season coincides with the dry season and it is therefore necessary to use the water that has been stored up in the snows of the high mountains during the winter for artificial application to the crops in the summer. It is estimated that from 60 to 90 per cent of the total annual precipitation on the high watersheds in Utah falls as snow and accumulates throughout the winter, forming the storage supply from which water is drawn in the summer.

Only 20 per cent of the area of Utah is more than 7,000 ft. above sea level, yet this 20 per cent contributes from 70 to 80 per cent of the total run-off. The accumulated snow cover on the high watersheds may therefore be considered as a huge natural storage reservoir.

Recognizing the importance of a better knowledge of the water-producing areas of the state, the Utah Agricultural Experiment Station in 1923 undertook a study of the relationship between the precipitation on, and the run-off from, the high watersheds of the state. Such a study required measurement of precipitation and collection of other meteorological data on the watersheds, and measurement of run-off from the watersheds.

RELATIONSHIP BETWEEN SNOW COVER AND RUN-OFF

Each stream is apparently a law unto itself, and the relationship between snow cover and run-off must be ascertained for each individual case. In forecasting the run-off on some watersheds, snow cover is the only factor that needs to be considered, but on others watershed characteristics and spring and fall rains may have a material effect. Another disturbing factor that may be present in certain years on all watersheds is the premature melting of the snow cover at low elevations.

The percentage method, developed by Dr. J. E. Church of the University of Nevada, forms the basis of the water-supply forecasts in Utah. By measuring the snow cover for a number of years and plotting the data obtained against run-off, the basic relationship between snow cover and run-off can be determined. This relationship must be modified for any particular year if the factors (other than snow-cover) affecting run-off are abnormal. Watershed characteristics will largely determine the relationship between snow cover and run-off, and it is for this reason that the snow cover on one watershed may not be indicative of the run-off from another watershed. Once the basic relationship and the effect of the modifying factors for a particular stream have been determined, a measurement of snow cover at the end of the precipitation season will indicate the probable water supply during the following summer months. The longer the record, the more accurate the determination of the relationship between snow cover and run-off.

CHARACTER OF PRECIPITATION

Winter precipitation in the West is largely the result of cyclonic storms. In spite of the widespread character of these storms, there seems to be no definite relationship between precipitation in the valleys and that on the adjacent mountains. Six years of measurements at valley and mountain stations in Utah indicate that precipitation in the valleys during the winter was only 55 per cent of the total annual precipitation, while winter precipitation in the mountains equaled over 90 per cent of the annual precipitation there. These data indicate that the measurement of precipitation at valley stations is not a good index of the potential water supply. In an attempt to forecast water supply, the first prerequisite is a measurement of the precipitation on high watersheds.

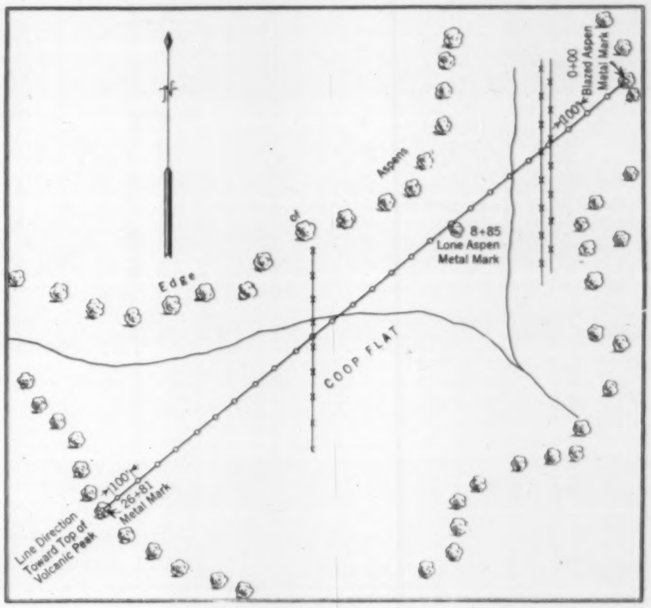
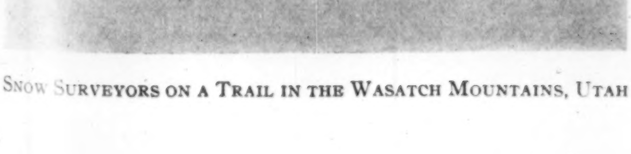


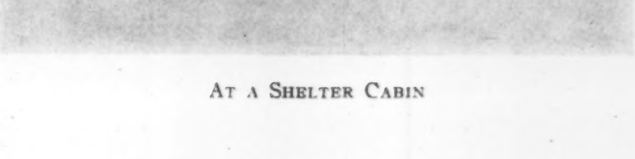
Fig. 1. Typical Snow-Course Layout
Coal Creek Basin, Utah

During the winter, the snow that falls and accumulates at high altitudes loses little or no moisture by melting. A measurement of this snow cover at the end of the winter season, before melting begins, is a good index of the potential water supply for the following season. At present the only feasible method of obtaining the needed information is by making snow surveys at the end of the precipitation season, which in Utah is about April 1. The spring run-off usually starts some time between April 10 and May 1. Precipitation that falls after the date of the snow survey and before the peak of the run-off is reached, if abnormal in amount or distribution, will affect the basic relationship between snow cover and run-off.

As early as 1800 it was recognized that snow cover plays an important part in the amount and time of run-off, but it was not until a hundred years later that



Snow Surveyors on a Trail in the Wasatch Mountains, Utah



At a Shelter Cabin

methods of measuring this cover were developed and used to any extent. The density of the snow varies widely from year to year, so that depth measurements alone are not indicative of the potential water supply in the cover.

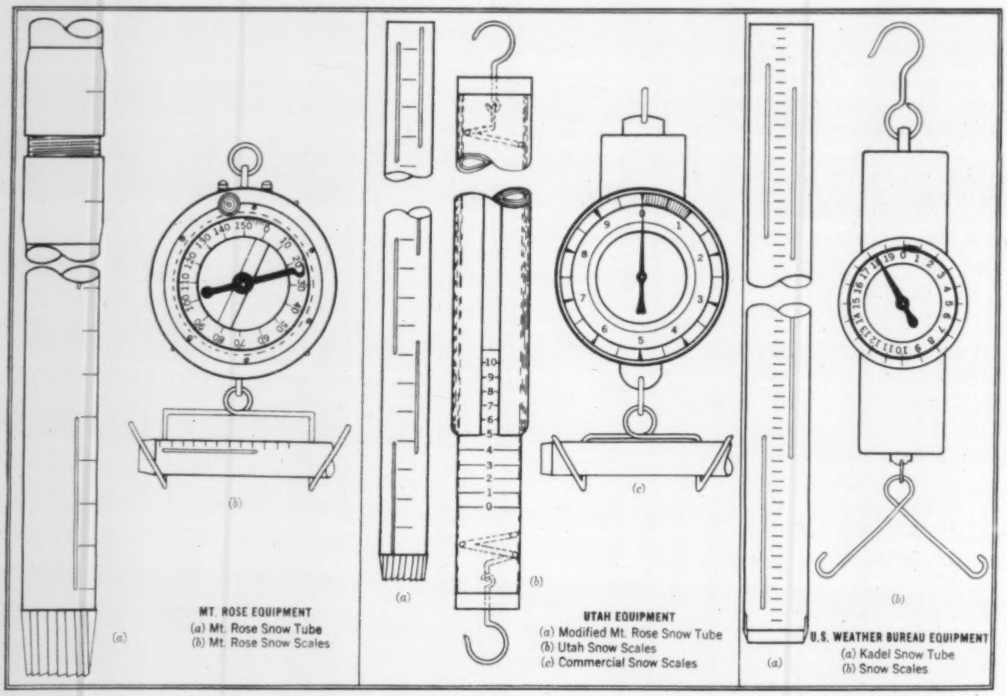


FIG. 2. TYPES OF SNOW SAMPLERS AND SCALES IN USE IN UTAH

The idea of weighing a snow core of known volume to determine its water content was first developed by R. E. Horton, M. Am. Soc. C.E., in his work in 1903 and 1904 in New York. The pioneer in measuring the snow cover in Utah was J. C. Alter, of the U.S. Weather Bureau. He used a steel tube for cutting out a core of snow, which he weighed to determine its water content. In 1909 Dr. Church developed equipment for cutting out a known volume of snow and weighing it to determine its water content. This method is accurate, and the equipment can be conveniently handled in the field. His is now the standard equipment used for determinations of water content.

In his study of snow cover, Dr. Church found that the depth and water content of the snow cover on a given watershed could best be determined by a system of surveys over representative sections of the watershed instead of by measurements taken at random. These representative surveys are made by measuring the depth and water content of the snow cover at regular intervals over established courses, located across representative areas and carefully described and marked so that the measurements can be made in the same places each year. The courses, which vary in length from 200 to 5,000 ft., are located so as to be as free as possible from drifting winds and above the line of winter melting. Measurements are made at 50 or 100-ft. intervals, depending on the length of the course. The line of winter melting in Utah ranges from an elevation of 7,000 to 9,000 ft. depending on the geographic location. Some courses should be located below the line of winter melting to determine the fluctuation and amount of low snow. In Fig. 1 is given a typical snow-course layout.

The Mt. Rose snow sampler, as developed by Dr. Church, consists of a seamless steel tube having an outside diameter of 1.75 in. The cutting edge in the end has an internal diameter of 1.5 in. The tube is slotted on alternate sides to enable the observer to determine the length of the snow core and to facilitate its removal. These cutters, which are made of high-carbon steel, have a serrated edge and a spiral milling to aid in penetrating snow crusts and ice. A shoulder in the cutter prevents the core from sliding out when the tube is being withdrawn from the snow. The tube is graduated in inches on the outside, beginning at the cutting edge.

The Mt. Rose scale, used to weigh the snow core, is of the spring-balance type and has a circular dial and a pointer. The pointer may be set to zero with the empty tube on the scale, so that the water content in inches can be read directly. The equipment used in Utah is the same as that of Dr. Church, except that the diameter of the core is changed so that a depth of water of 1 in. in the tube weighs one ounce. With this tube may be used ordinary commercial scales reading to ounces. In addition to the tube and scales the surveyor must have a tape, a field book, a map, a cloth for cleaning the instruments, a leather thong, a hatchet, a barometer, a thermometer, a compass, and snowshoes or skis. The various types of equipment used in the Utah cooperative snow surveys are shown in Fig. 2.

A survey party should never consist of less than two men. On long trips a third man is necessary. It is of no advantage to have more than three men in a party. The field procedure for making measurements of depth and water content may be outlined as follows:

1. Beginning at the initial point, measure the distance to the first observation point, as shown in Fig. 1.
2. Stick an alpenstock or a forked stick into the snow and suspend the scales from it.
3. Weigh the tube empty and record its weight in the field notes.
4. Hold the tube vertically with the cutter against the snow and force it downward with a steady thrust, twisting only when ice crusts are encountered. The operator can tell when the bottom is reached by the feel. Care must be used, however, when working over rocky ground, since the rock and the ice feel about the same. If the bottom is soft, push the cutter into the mud about 0.5 in. before withdrawing the tube.
5. Record the depth of snow.
6. Twist the tube to break the core loose from the ground.
7. Withdraw the tube by pulling vertically upward with a steady motion. Jerking will dislodge the core. If the tube sticks, twist it slightly backwards (to the left).
8. Check the length of the core. If the core length varies from the depth by more than 6 in., take another sample.
9. Hold the tube in a horizontal position and examine the cutter end. If there is mud in the cutter, clean it out carefully and record its depth. To determine the true core depth, the depth of mud must be deducted from that of snow.
10. Lay the tube in the sling attached to the scales, taking care to keep it in a horizontal position so that the core will not slide out.

11. When the spring of the scale has come to rest under the load, shake the spring slightly to prevent it from sticking. Record the scale reading.

12. To dislodge the core, invert the tube and shake it slightly. Hold the tube up from the snow so that the core will not lodge in the end of the tube as it comes out. If the core sticks, jar the end of the tube on the framework of a snowshoe or ski. If the core freezes in the tube, as it does under certain conditions of temperature and density of snow, it may be necessary to burn a few newspapers or a candle under the cutter to thaw it out. The tube should then be cleaned out with a dry rag. Snow will stick in the tube much less readily if the tube and the cutter are kept well shellacked, and if the cutter and the lower end of the tube are kept free of mud.

Areas where snow surveys must be made are usually uninhabited, and adverse weather conditions may be encountered at any time. The surveyors should be properly equipped with snowshoes or skis and suitable clothing, dark glasses, and face masks. Shelters should be provided at intervals of not over 10 miles, or, if travel is difficult, not over one day's hike under adverse weather conditions. These shelters should be stocked with blankets and with provisions packed in tight metal containers.

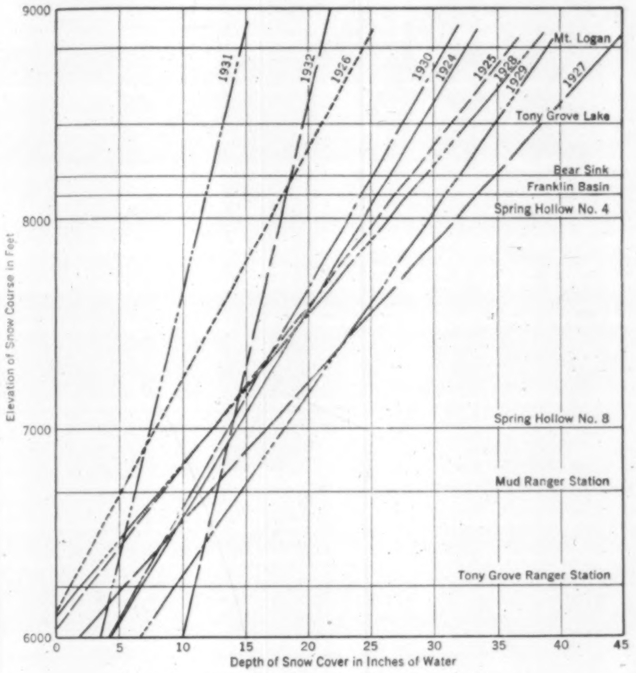


Fig. 3. Snow Cover and Elevation
Logan River Watershed, Utah

Snow surveying in high, rough, and inaccessible areas is hazardous at best, and surveyors should avoid undue exposure and travel during blizzards, and should be continually on the alert for snow slides. In case of doubt as to weather or travel conditions, surveyors should remain in the shelters.

LOGAN RIVER STUDIES

Detailed studies of the relationship between snow cover and run-off have been made on the Logan River watershed in Utah for the past eight years. When this work was started, no data on snow cover were available except a few isolated stake readings collected by the U.S. Weather Bureau and the U.S. Forest Service. The first job, therefore, was to establish snow surveys for the determination of the water content of the snow cover at the end of the precipitation season. Stream-flow records for approximately 35 years are available on the Logan River.

The watershed of the Logan River covers approximately 220 square miles and varies in elevation from 4,700 to 10,000 ft. It is about 25 miles long, north to south, and 9 miles wide, east to west. The maximum recorded seasonal discharge is 370,000 acre-ft. and the minimum is 92,800 acre-ft. The topography is rugged and the formation is largely limestone, which is badly broken. Three regular snow-survey courses were established at representative points above an elevation of 8,000 ft. and a number of shorter courses between elevations of 6,000 and 8,000 ft.

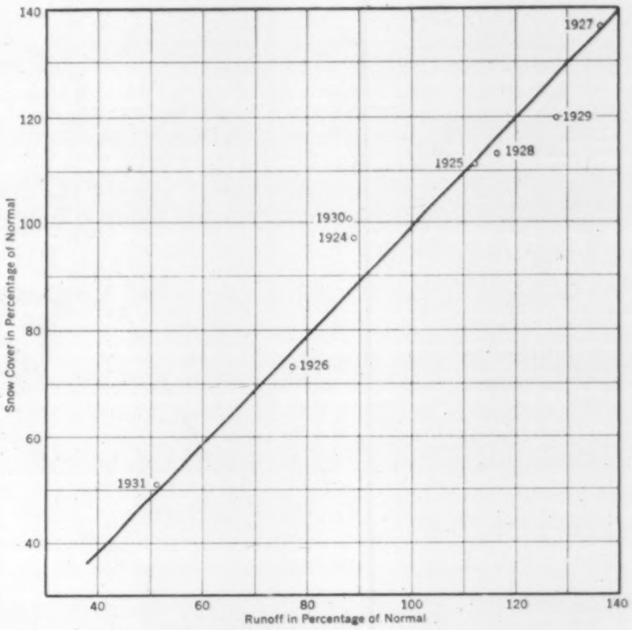


Fig. 4. Relation Between Snow Cover and Run-Off
Logan River Watershed, Utah

It was found that the snow cover above 7,000 ft. was a better index of run-off than that at lower elevations on account of winter melting. Therefore, in the analysis on the Logan watershed, only the three snow courses above 8,000 ft. are considered. They represent in the aggregate a total of 106 measurements at 100-ft. intervals. Each of these three courses is protected from abnormal drifting and represents the snow cover on adjacent areas.

Monthly snow surveys are made up the side of Mt. Logan to check the accumulation of the snow and to determine the variation of water content with elevation. In Fig. 3 are plotted, for each year, the variation in water content of the snow cover with elevation. This variation is wide and there appears to be no definite relationship between elevation and water content. In 1927 the increase in water content with elevation was 13 in. per 1,000 ft., and in 1931 it was 3.5 in. per 1,000 ft. The 1932 curve parallels the 1931 curve and shows that, although precipitation in 1932 was much heavier in the valleys, it was not proportionately heavy in the mountains.

During the period of 1924–1931, inclusive, the average water content of the snow cover at elevations above 8,000 ft. on the Logan watershed has varied from 14.4 in. in 1931 to 39.6 in. in 1927. To determine the basic relationship between snow cover and run-off, the snow cover, in percentage of its normal, was plotted against the run-off for the period desired, in percentage of its normal. The high-water run-off on the Logan River usually begins during the latter part of April or the first part of May and lasts sometimes until July 1. The low-water period on the river is from July 1 to September 30. After plotting the snow cover against the run-off for many different periods, it was found that the closest correlation between the two was for the period between July 1 and September 30, inclusive. This means that the high snows are the principal source of the late summer water supply.

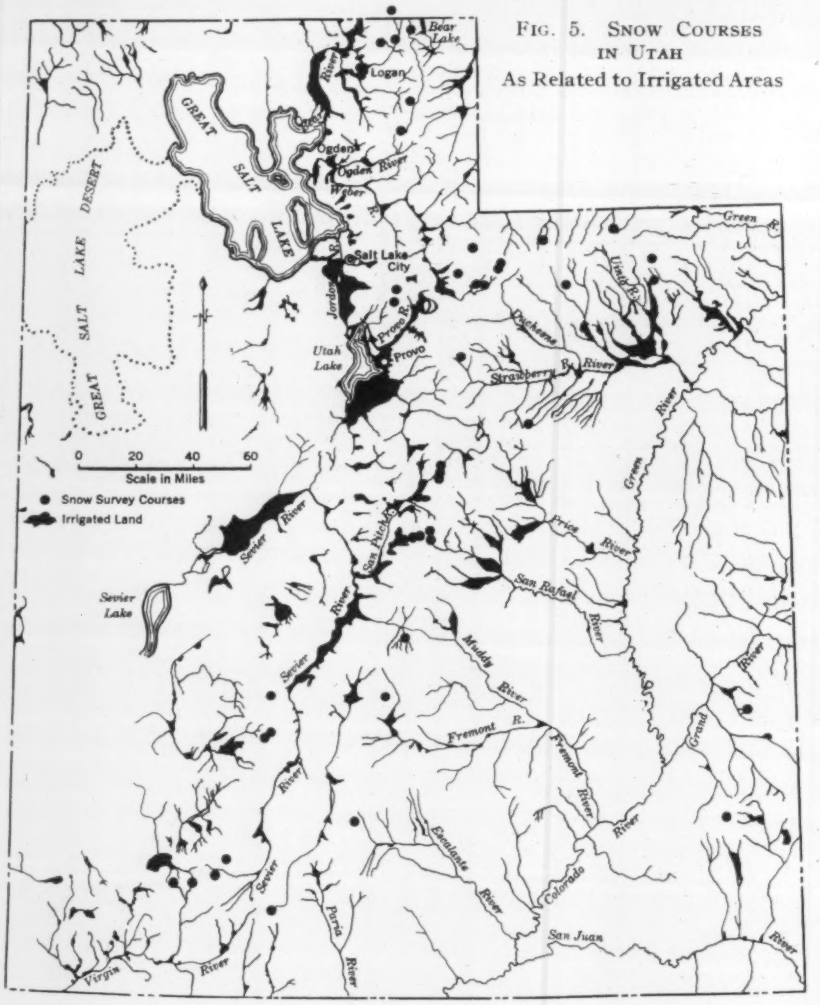


FIG. 5. SNOW COURSES IN UTAH
As Related to Irrigated Areas

The water content of the snow cover, in percentage of its normal, is plotted in Fig. 4 against the run-off between July 1 and September 30, in percentage of its normal. All the points do not fall in a straight line, but only two points miss it by as much as 5 per cent and only one misses it by as much as 10 per cent. It is recognized that the record is short, probably too short to indicate the basic relationship between snow cover and run-off for this stream. However, the graph indicates that there is a definite relationship between high snow cover and the run-off. It will be noted that the years of 1924 and 1930 are off the curve. In both years the run-off was less than that indicated by the snow cover. This may be due to several factors: deficient precipitation in the spring; abnormally low temperatures during the winter; deficient moisture in the soil at the beginning of the precipitation season; or premature melting of the low snows. Sufficient data are not yet available to make possible the evaluation of these modifying factors. As they become available it will no doubt be possible to explain the divergence in run-off of any one year from that indicated by the basic relationship. For this particular stream, it is now possible to predict the run-off between July and September to within 10 per cent. The forecast made on April 1, 1931, was well within 5 per cent of the actual run-off. Considering the length of the record of snow cover and the further fact that only one of the many factors affecting run-off is considered, this accuracy is within reasonable limits.

UTAH'S STATE-WIDE SNOW SURVEYS

Snow surveys were started on the Logan River watershed in 1924, and those on Coal Creek, near Cedar City, and on the Sevier River, near the Kimberly Mine, were begun in 1925, with the assistance of the local water commissioners. As a result of an informal cooperative agreement between the Utah Agricultural Experiment Station, the U.S. Weather Bureau, and the U.S. Forest Service, a network of snow surveys covering the principal watersheds of the state was established in 1930. The snow surveys are made about April 1 of each year by the U.S. Forest Service, by local water commissions, by the Utah Agricultural Experiment Station, or by other cooperating agencies. The courses were located and established by me with the assistance of the cooperating agency of the particular locality. The equipment and methods previously described are used on all these courses. The map of Utah given in Fig. 5 is marked to show the location of the snow courses in relation to the irrigated areas of the state. The network consists of about fifty courses, most of them above 8,000 ft. in elevation. One year's record is now available on all these courses, and on more than half of them measurements have been taken for two years.

These surveys give a fair cross section of the principal high watersheds of the state; more courses are needed, however, to furnish detailed data for the small watersheds. All irrigation companies are interested in the snow cover, which is the source of their water supply. By means of a snow survey of their watersheds at the end of the winter season it is possible to determine the probable water supply before the crops are planted.

Results of the state-wide snow surveys are published annually by the U.S. Weather Bureau in the March (monthly) Climatological Data for Utah. This publication is available for public distribution about April 15. The snow-cover measurements are published by the local papers immediately after the surveys are completed. The data may also be obtained from the U.S. Forest Rangers or local water commissioners making the surveys. Water-supply forecasts are issued about April 10 of each year for the Blacksmith Fork and Logan rivers. A general statement of the condition of the snow cover throughout the state is also issued at that time. As soon as sufficient records are available, water-supply forecasts will be made annually for the principal watersheds of the state.

Present-Day Transportation Problems

Interrelation of Trucks, Buses, Waterways, Highways, and Railroads

By J. S. Worley

Member American Society of Civil Engineers

Professor of Transportation Engineering, University of Michigan, Ann Arbor

When it is stated that transportation is our second largest industry, that the total investment in it exceeds fifty billion dollars (more than one sixth of our commercial wealth), and that our present industrial system is built around, and is dependent on it, nothing further need be added to bring about a realization of the magnitude of the subject. Professor Worley points out that, from the standpoint of the taxpayer, every waterway project constructed with tax funds should have charged against it an amount sufficient to balance the increased rail freight rates made necessary by the loss of business to the railroads. Other pertinent phases of the transportation industry in this country are presented in this article, which is based on a paper presented by the author before the Detroit Section of the Society last winter.

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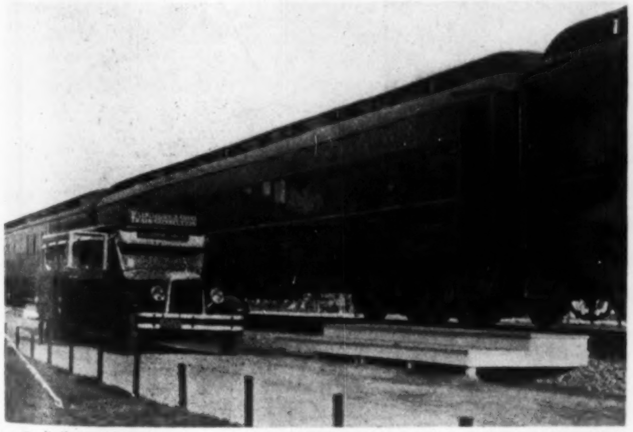
As a rule, common-carrier transportation agencies are private property, privately owned and privately operated for the sole purpose of private gain. The interest of engineers, however, is purely of a public nature; that is, they are concerned with the common or public good. This principle of public interest or public welfare is not to be interpreted to mean public gain at the expense of private property. Public gain at private expense or private gain at public expense can end only in failure.

The transportation problems of the common carriers are the same today as they have always been. The public is interested in three things: the quality of the services rendered; the political aspects of the problem; and the charges made for, and the cost of, the services rendered. The item of quality of service may be subdivided into four parts: capacity to serve, dependability, safety, and discrimination in charges and service. Under political aspects may be listed the following factors: restraint of trade, pooling, control of legislation, corruption of the judiciary, corruption of public officials, and state control or direction of certain administrative activities of the carriers. The charge for services rendered is also of vital importance and must be in excess of the cost of rendering the service, otherwise the business will not be successful.

Every business enterprise, to be considered successful, must earn on an average a sufficient annual income to meet its operating expenses, to set up a proper depreciation reserve, to pay its taxes, to meet other expenses due to legal requirements, and to pay a return on a specific sum of money. Any business, a private or a public utility, that does not have sufficient income to meet these requirements will ultimately fail.

It is to the public interest that charges for transportation service be low, but it is of equal importance to the public that the charges be sufficient to make the business a success under efficient and economical management. In an effort to secure conformity with these principles, the states and the Federal Government have enacted laws and created public utility commissions and the Interstate Commerce Commission.

The principal transportation problems now confronting us are as follows: (1) regulation of common-carrier transportation by bus and truck; (2) development of the St. Lawrence Waterway; (3) development of inland navigation, principally that of the Mississippi River and its tributaries; (4) construction of the Intracoastal Waterway from Boston along the Atlantic coast to Florida, across Florida to the Gulf of Mexico, and along its coast to Brownsville, Tex., on the border of Mexico; (5) a more efficient planning, designing, and adminis-



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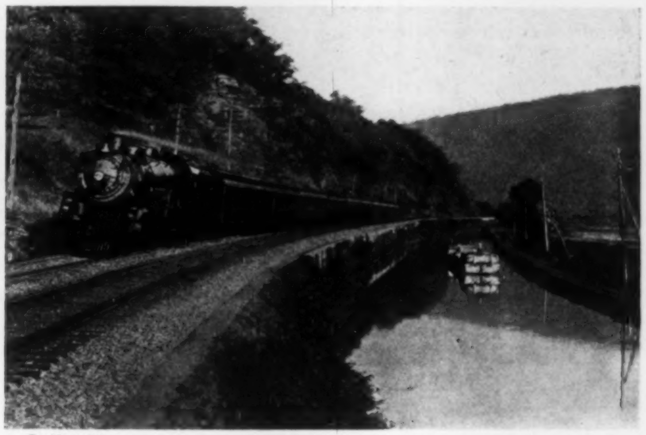
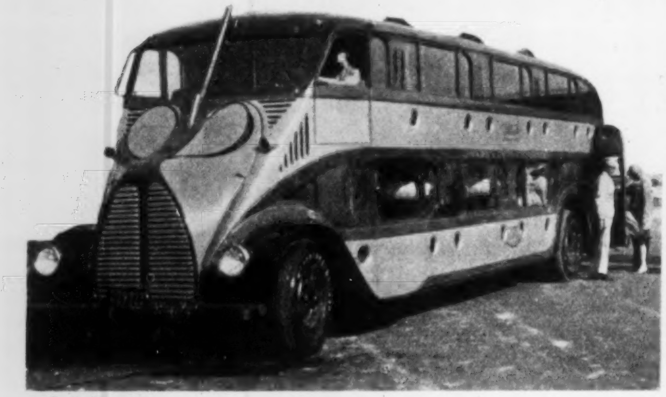
Bus Makes Connection Between Baltimore and Ohio Railroad Terminal at Jersey City, N.J., and New York City



© Nesmith

Representatives of Two Forms of Transportation Pass Each Other in the Potomac River Valley tration of our trunk-line highways and secondary roads, both county and township; (6) expenditure of public funds for, or in aid of, transportation facilities, the public benefit from which may not warrant such outlay;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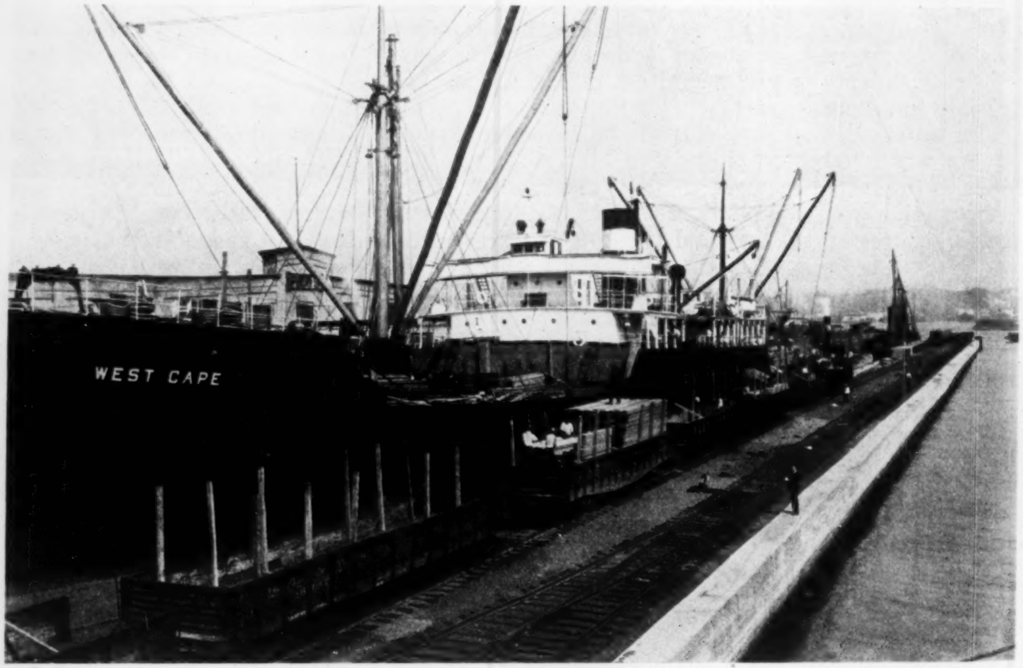
A RECENT DEVELOPMENT IN BUSES—THE NIGHT COACH

and (7) the present charges for railway services, which are not sufficient for a successful business enterprise. All these problems will be considered briefly as they pertain to the public interest.

FURTHER REGULATION OF BUSES AND TRUCKS

Public utility commissions of the various states are at present regulating intrastate service of common-carrier bus and truck transportation. The Federal Government has not promulgated any regulation with reference to these carriers. Public interest in the matter is very largely dependent on the service rendered to users. Since this service is reasonably satisfactory, there is some question as to whether public interest will be served by an intensive and elaborate system of regulation. In the last two or three years there has been considerable discussion in the newspapers and periodicals of the country and some agitation for such regulation. This agitation has not come from the users of buses and trucks but from the railroads, because they have lost business to this new form of transportation.

The railroads have lost between 20 and 25 per cent of their passenger business, a very large part of which is now being carried by private cars. According to the latest and most authoritative sources, the amount being carried by common-carrier buses, exclusive of intracity service, is 10 per cent. As regards freight business, the railroads have lost directly to the automotive truck somewhat less than 5 per cent. In addition to the amount of business lost directly to the truck, the railroads have lost a considerable amount of business in carrying coal because of the introduction of oil and natural gas as fuel. This loss amounts to something less than 5 per cent. The total amount of freight business thus lost by the railroads during the last five or six years is less than 10 per cent.



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RAIL AND WATER TRANSPORTATION MEET

Lumber Unloaded Directly from Steamer to Car at Hoboken, N.J.

PROTECTION OF COMMON CARRIERS

The public is interested in the regulation of truck transportation by contract carriers which, because of the nature of their business, have been able to avoid the jurisdiction of state public utility commissions and have not paid the taxes required of common carriers. It is to the public interest to promote and protect common-carrier transportation companies. If contract carriers engage in transportation business to the detriment of the common carriers, the practice destroys the business of the common carriers and is against public interest.

The public may find it to its interest to impose a further road tax on common-carrier motor vehicles, or to restrict their use of the highways, but this is a matter foreign to the subject of this article.

WATERWAYS INCREASE RAILROAD FREIGHT RATES

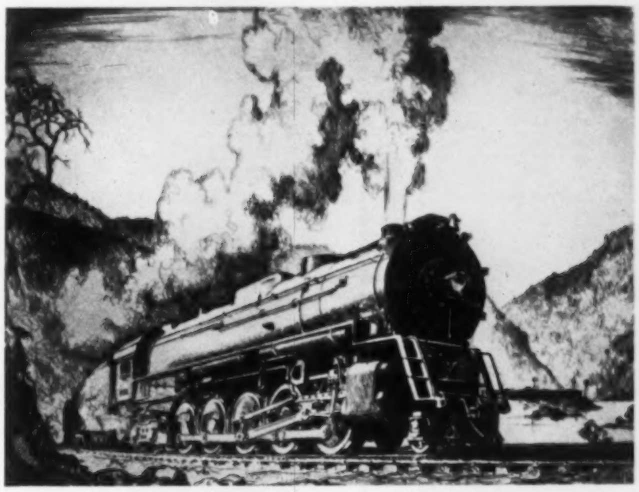
Providing that the benefits warrant the expense, the development of the St. Lawrence Waterway will further the public interest. According to the present plan, more than half the cost will be charged against water power development, which will be of advantage to the public. It would appear that the remaining expense, that charged to transportation, whereby river towns in the United States and Canada will be made ocean ports, will warrant the carrying out of the project. From the point of view of the public interest, there must be charged against this enterprise not only the capital cost and the interest on it, and the cost of repairs and maintenance, but also the increase in freight rates made necessary on our railroads by the loss of the transportation business that will be diverted from them to the waterway.

The development of a system of inland transportation by water has received considerable attention during President Hoover's administration. One of the President's first pronouncements was on the development of inland waterways. These waterways will be paid for and maintained from taxes, and can only be justified by a reduction in freight rates sufficient to pay the capital expenditures for construction and the annual maintenance costs. In addition, there must be a further saving in freight rates by water sufficient to cover the increase in railway freight rates that will be required to maintain these carriers, because of the business they will lose. A preliminary study of this subject suggests that the value of the enterprise, exclusive of the Great Lakes section, is questionable.

The Intracoastal Waterway, extending from Boston down the Atlantic coast and across Florida to the Gulf of Mexico, and along the Gulf to the Mexican border at the mouth of the Rio Grande, is a project about which little is heard except along the Atlantic seaboard. It will furnish barge transportation in canals and waterways built back of the outlying islands, shoals, and other pieces of land along the coast in a canal constructed across New Jersey and another across Florida. The expenditure will be very large and will be paid for entirely with taxpayers' money. From the standpoint of public interest, the enterprise is of questionable value.

HIGHWAY DESIGN AND FINANCING BOTH NEED STUDY

At the present time there is great need for a scientific study in connection with the extension of trunk-line highways and the improvement and administration of secondary highways (those primarily known as county and township roads). Up to the present time, state trunk lines have been planned largely to meet immediate demands. In location, alignment, grading, and other major phases, they are not able to meet efficiently the demands now made on them. In addition, the surfaces, mainly concrete slabs, have been of one design, and very little attention has been given to the amount or kind of traffic served. There is need for a scientific study of the planning, locating, and designing of our roads so that they will be better adapted to the traffic passing over them. Careful financial planning is also needed as regards the construction, maintenance, and future reconstruction of the highways. There should also be better coordination between state, county, and township roads.



A Pay Load
From an Etching by Otto Kuhler for the Lima Locomotives Works

The expenditure of public funds obtained from taxation, bond issues, or similar sources, for the expansion of transportation facilities—that is, for the building of highways and airports, for the improvement of inland waterways and harbors, and for similar projects—should be very carefully studied to ascertain whether real public interest will be served by these expenditures. This is a problem of very great import, which should have the close attention of all citizens.

PAN AMERICAN AIRWAYS SYSTEM

© Nesmith

The Last Word in Aerial Transportation
The *American Clipper* of the Pan American Airways System

RAILROADS NEED AID

Our most serious transportation problem today is that of the railroads. Almost 26 billion dollars are invested in the railroads of the country. Exclusive of shipping on the Great Lakes, the railroads handle 90 per cent of the common-carrier freight of the country. Our manufacturing and industrial systems, including agriculture, have been created according to the pattern of the railroads, and cannot continue to exist unless these particular transportation facilities are available. The cessation of railroad operation would bring about disastrous results, for it would wreck the whole economic and financial structure, would cause great poverty and distress, and in many areas famine and death. It would set back the political, economic, educational, and other development of the country by several hundred years. To do away with automotive, inland-waterway, and aerial transportation would cause some inconvenience, but to do away with the rail transportation system would be a major catastrophe.

© Welland Ship Canal, Canada

Flight of Locks on the Welland Ship Canal
Between Lake Ontario and Lake Erie

PRINCIPLE OF CONTROL POLICY

About 1800, at the beginning of the present era of public utilities, which came into existence some time before the railroads, a policy of regulation and control was adopted. This policy, which was proper and in the public interest, has been so extended in the case of the railroads that at this time governmental control affects practically their every activity, including the amount, character, and physical condition of the equipment used; the schedule of trains operated; general administration; hours of labor; composition of train crews; safety devices; signaling systems; accounting methods; purchase of materials and supplies; merging and consolidation of companies; award of contracts; rates and fares to be charged; plan of construction and financing; reconstruction and refinancing; and the rate of return that the companies are to enjoy.

This control has been tightened to such a degree that during prosperous times the railroads are not permitted to benefit from the prosperity enjoyed by other forms of industry. Under this plan of regulation it was expected that when the properties were not permitted to enjoy the full benefit of prosperous times they would receive some protection during less prosperous times by an adjustment in rates so that they would have at least a nominal return. The plan had as its object the stabilization of the investment, of the return, and of the quality of service rendered, so as to ensure adequate transportation facilities for industry. Such a policy is in the public interest.

RAILWAY SERVICE IS SATISFACTORY

As regards quantity and quality of service; honest dealing with patrons; courteous treatment of the public; and economical and efficient construction, financing, and operation of properties; no criticism can be made and here there is no railroad problem. There is a very serious problem, however, in that the railroads are not permitted to share in the prosperity of the country during good times, so that they cannot lay aside a reserve for periods of depression. The Government did not adhere to the principle that rates should be adjusted to protect the investments of the railroads and keep their properties intact. As a result the railroads now find themselves in a serious financial condition.

During the last three years, the railroads have not earned $5^3/_4$ per cent on their first cost, which is the minimum value to which the $5^3/_4$ per cent should be applied. During this period, expenditures of every kind have been curtailed to such an extent that maintenance of equipment and properties has been, and still is, seriously neglected.

MEAGER RETURNS AND DEFERENCE OF NEEDED MAINTENANCE

It is probable that the returns for 1932 will be 1 per cent or less. Such curtailing of expenses and deferring of needed maintenance and repair work as has taken place during the last three years cannot continue indefinitely. Many of the roads are now faced with receivership; some of them have recently gone into bankruptcy. All this is due to the fact that the Government has gone only halfway in conforming with the principle of regulation; the rates have been adjusted so that the railroads will not receive large returns in good times, but have not been regulated correspondingly so that they will receive a fair return during bad times. This condition is having a serious effect on banks, trust companies, insurance companies, and others that have invested their money in the securities of these railroads in the faith that the plan of regulation would be carried out and their investments protected.

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Steamship Leaving Lock No. 7
On the Welland Ship Canal Around Niagara Falls

Under receivership management and operation, it will be only a short time before the physical condition of these properties will be serious, and industry will be deprived of the transportation service necessary for its prosperity. The condition of the railroads, under past and present regulation, is such that they are not able to efficiently serve the public interest. There is a serious railway transportation problem.

Rudimentary Economics of Highway Design

Effect of Gradient, Alignment, and Surfacing on Costs of Vehicle Operation

By W. W. Zass

Member American Society of Civil Engineers

Engineer of Construction, Arkansas State Highway Commission, Little Rock, Ark.

MORE and more, highway engineers are considering the problems of highway location and construction from the point of view of the man behind the wheel. As a result, one hears much of savings in the cost of motor-vehicle operation. In this article, Mr. Zass takes material available in other sources and presents in readable form an informative discussion of the relations between the costs of automobile operation and road construction and maintenance. All engineers, whether highway specialists or simply highway users, will find food for thought in this article.

WHEN the changes in gradient are slight and compensating—that is, as much plus grade as minus grade—when there is little curvature, and the surfacing is of a high type, a highway may be said to be ideal from the point of view of the vehicle-operating public. This is because the three major factors affecting the cost and comfort of motor-vehicle operation are gradient, alignment, and type of surfacing. A change in any or all of these factors may materially increase or decrease the expense of vehicle operation.

In preparing a modern highway program, careful attention is given to the location of the road to be improved so as to secure between objectives the shortest possible route consistent with satisfactory alignment and gradient. In all but a few instances these two factors, together with the item of first cost, make it impossible or impracticable to connect objectives by the shortest possible route, that is, by a straight line. Deviations in alignment are due mainly to the effort to evade obstacles, either natural or artificial, without exceeding a reasonable first cost. Changes in elevation are the result of the effort to make as few changes as possible in alignment.

Changes in gradient that are compensated and small enough so that a change of gear is unnecessary, generally involve no additional operating expense. Considering all classes of motor vehicles, trucks, buses, and passenger cars of all weights, and the probable percentage of each class traversing the highway, it is considered that the ruling grade should not be more than 7 per cent. Where bus and heavy truck traffic is predominant, it is probable that the ruling grade should not exceed 4 per cent.

ROLLING RESISTANCE

As a result of research—notably tests conducted by Iowa State College, some of the conclusions of which are described in the college Bulletins Nos. 1, 67, and 88—it may be said that the rolling resistance developed by a motor vehicle varies with its weight and speed and the type of surfacing over which it travels. This rolling resistance, or resistance to movement, is made up of the frictional resistance of the parts within the vehicle, the friction between the wheels and the road surface, and the normal air resistance.

It has been determined experimentally that the total rolling resistance varies from about 20 lb. per ton of weight of vehicle and load, for vehicles in excellent condition traveling on high-type surfacing, to approximately 140 lb. per ton for those in a fair or poor condition traveling on inferior surfaces. In Fig. 1 are shown some of the results obtained at Iowa State College. Assuming a normal value for rolling resistance of 100 lb. per ton of weight of vehicle and load, the amount of energy required to move one ton of vehicle load 1 ft. can be computed as 100 ft-lb., 1 ft-lb. being the energy required to move a weight of 1 lb. through a distance of 1 ft. on a level surface.

A motor generating sufficient power to propel a vehicle at a speed of 25 miles an hour against such resistance has developed in the vehicle a kinetic or moving energy proportional to its weight. Assuming a total load of 2,000 lb. and a speed of 25 miles per hour, a kinetic energy of 41,900 ft-lb. is attained by the vehicle.

It has been previously noted that a vehicle might be expected to develop a rolling resistance of 100 lb. per ton of gross weight. Therefore, if the motor is shut off or disengaged after a speed of 25 miles an hour has been attained on a level grade, the vehicle will coast a distance equal to 41,900 ÷ 100, or 419 ft., if brakes are not applied.

Courtesy U.S. Bureau of Public Roads

Highway Location in Sequoia National Park

CLIMBING GRADES

The gravitational force encountered in climbing a slope can be similarly expressed. In addition to the rolling resistance previously computed, a rise of 1 ft. in 100 ft. for a loading of 2,000 lb. would entail work to the amount of 2,000 ft-lb. for the foot of rise surmounted. Therefore the energy required for climbing the vertical distance of 1 ft. in 100 ft. would be 2,000 ÷ 100, or 20 ft-lb.

The total resistance developed in climbing a 1 per cent grade is equal to the gravitational plus the rolling resistance, which for the example cited would amount to 20 + 100 or 120 ft-lb. per lin. ft. of distance traversed for each ton of loading. Since at a speed of 25 miles per hour, a kinetic energy of 41,900 ft-lb. is developed if the motor is shut off when a vehicle traveling at this speed is ascending a 1 per cent rise, the vehicle will travel a distance of 41,900 ÷ 120, or 349 ft. before coming to a stop, if brakes are not applied.

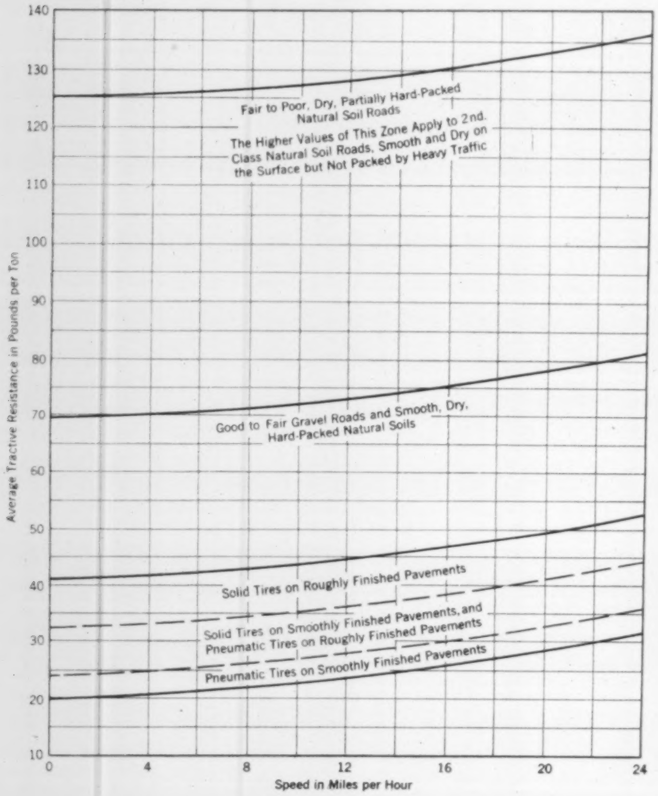


FIG. 1. RANGE OF VALUES OF ROLLING RESISTANCE, INCLUDING NORMAL AIR RESISTANCE
From Bulletin No. 67 (1924), Engineering Experiment Station of Iowa State College

BALANCING DISTANCE AND GRADIENT

The principles described indicate that the total work expended per ton of loading for each linear foot of distance traversed is 20 lb. per ton greater for each rise of 1 ft. in 100 ft. than for the same distance traversed on the level. The relationship between the resistance to motion developed on a level and that developed on an ascending grade may be expressed as:

$$\frac{RT}{RT + 20\,rT} \quad \cdots\cdots \quad [1]$$

in which R equals rolling resistance, T equals gross load in tons, and r equals rise in feet per 100 ft. of distance, or rate of grade.

This expression indicates the balance, or economic mean, required between distance and gradient. As an illustration it may be assumed that the distance between Points A and C via Point B is 5,000 ft. and that a level route could be constructed through these three points. An alternate route only 4,000 ft. long could be selected between Points A and C, via Point D, but some gradient would be required. Using the values previously assumed, the energy expended for each ton of vehicle loading will be 5,000 × 100 × 1, or 500,000 ft-lb. per ton for the route ABC, and $4{,}000 \times 100 \times 1 + (20r \times 1)$ or $(400{,}000 + 80{,}000r)$ ft-lb. for the route ADC. The equivalent grade on route ADC to provide the same total resistance as was developed in traversing route ABC is indicated by equating these values

$$400{,}000 + 80{,}000r = 500{,}000, \text{ or } r = 5/4$$

A total rise in gradient of $5/4 \times 4{,}000 \div 100$, or 50 ft. in the distance ADC, would provide a similar total resistance to motion through 4,000 ft. of distance as would be encountered through 5,000 ft. of level grade. In so far as the total work performed is encountered, the climb of 50 ft. could be accomplished through a linear distance of 500, or 1,000, or 3,000 ft. Vehicle operating costs for fuel in the example cited should be the same for either route. For all other costs involved, the shorter route should provide the greater economy, if the rise or rises in gradient are not so great as to require that the vehicle be operated in second or low gear.

SAVINGS IN VEHICLE OPERATION AND ANNUAL ROAD CHARGES

Assuming the cost of operation and replacement of an average motor vehicle to be 6 cents per mile, the traffic density required to offset an additional construction cost made necessary to provide a shorter route can be readily computed. For purposes of illustration, consider that Route A is two miles shorter than Route B but has the same rises and falls in gradient. With an average traffic density of 1,000 vehicles per day, the shorter route would give an annual saving in vehicle-operation cost of 365 × 1,000 × 0.06 × 2, or $43,800. If the interest and depreciation on the original investment, as well as the maintenance charges for the shorter route, in comparison to similar charges for the longer route, do not exceed $43,800 annually, a net saving will accrue through the choice of the shorter route. For example, consider that Route A requires the construction of 28 miles of road at a cost of $40,000 per mile, or a total cost of $1,120,000, and that Route B calls for the construction of 30 miles of work at a cost of $30,000 per mile, or a total of $900,000. Then the annual charge for each route will be as follows, assuming interest at 5 per cent, annual depreciation on the improvement at 5 per cent, and annual maintenance at $500 per mile:

ROUTE A		ROUTE B	
$1,120,000 at 5% =	$ 56,000	$900,000 at 5% =	$ 45,000
1,120,000 at 5% =	56,000	900,000 at 5% =	45,000
28 miles at $500 =	14,000	30 miles at $500 =	15,000
Annual charge . .	$126,000		$105,000

The annual charge for the improvement of Route A exceeds the annual charge for that of Route B by $21,000, but this amount is exceeded by the annual saving of $43,800 in the cost of motor-vehicle operation. Therefore it would be economical to construct the shorter route in so far as net saving is concerned. It would be well to emphasize the fact that it is the motor-vehicle operator who benefits by this saving and that the state, or other political subdivision, does not participate in it. In the case cited, the state pays a greater annual charge for the improvement in order that the owners of motor vehicles may secure an annual reduction in operating costs.

It may be assumed that there is a difference of 2 cents per mile between the cost of operating a vehicle over a high-type pavement and over a low-type surface. If there is a traffic density of 2,000 motor vehicles a day, the annual saving in vehicle operation secured through the use of a high-type pavement would amount to 2,000 × 0.02 × 365, or $14,600 per mile. The annual charges per mile for a high-type pavement would compare with those for a low-type surfacing approximately as follows:

	High Type		Low Type	
Interest on first cost	$25,000 at 5% =	$1,250	$8,000 at 5% =	$ 400
Depreciation	25,000 at 5% =	1,250	8.000 at 25% =	2,000
Maintenance	=	500	=	800
Total annual charge per mile		$3,000		$3.200

For the assumed annual charges, the annual cost of the high-type pavement is slightly less than that of the low type. Therefore the saving in vehicle-operating cost, for the assumed traffic density, would be clear gain, and a total saving of $14,600 plus $200, or $14,800 per mile, would be realized annually if a high-type pavement were constructed. Again it is to be noted that the state or political subdivision is not the recipient of the major part of this saving.

After analyzing the economies that can be secured through the proper location and design of a highway, the question arises as to why the best possible location and design are not adopted in all cases. The answer is that the principal factor affecting the problem is that of first cost. If sufficient funds are available an improvement can be constructed in the theoretically correct way, but if the funds are limited the construction must be adapted to the sum available.

MEASURING THE DAMAGE TO SURFACING

Thus far, improvements in alignment, gradient, and surfacing of highways have been discussed solely from the viewpoint of the motor-vehicle operator, and methods have been presented to determine the routing that will give the greatest saving in the cost of vehicle operation.

The maintenance cost of a completed highway is dependent to a large extent on the type and loading of the vehicles using it. The velocity of the vehicle develops centrifugal force tangential to the rim of the wheel and causes a vacuum as the tire is released from the surface, thus tending to displace the particles composing the surface and to cause transverse rolling or corrugation of such surfacing.

The weight of the vehicle and its load has a tendency to break down the surfacing, and the heavier loadings destroy its useful life in a much shorter time than the lighter ones The damage may occur because of poor foundation conditions, the impact of the vehicle on uneven surfaces, severe climatic conditions, or a combination of any or all of these causes. Also, braking of a vehicle often causes the wheels to slide over the surface and displace particles of the surfacing. The damage to the surfacing caused by braking is proportional to the gross weight of the vehicle and load; there is no variation due to speed except that a vehicle going at a higher speed usually slides for a greater distance. The same tendency to displace the surface particles is noted when a vehicle is set in motion. The force of starting friction may be considered as approximately similar to that of sliding friction, but the possible damage caused by starting friction is generally confined to a smaller area.

If a vehicle is moving at a speed of 25 miles per hour, it has developed a kinetic energy of 41,900 ft-lb., as previously computed. If brakes are applied and the wheels are immediately locked, the momentum will be sufficient to carry the vehicle some distance before the frictional resistance between the wheels and the surfacing brings it to a stop. Experiments conducted at Iowa State College, described in the College Bulletin No. 88 (see Fig. 2), indicate that the sliding coefficient of friction between the wheels and the surfacing as developed under such conditions, for all sizes of tires and for different types of surfacing, ranges from 0.50 to 1.0 per ton of vehicle and load. That is, for the normal vehicle having a gross weight of one ton, a sliding coefficient of friction of about 0.60 for gravel surfacing, and of 0.80 for high-type pavements, would be developed.

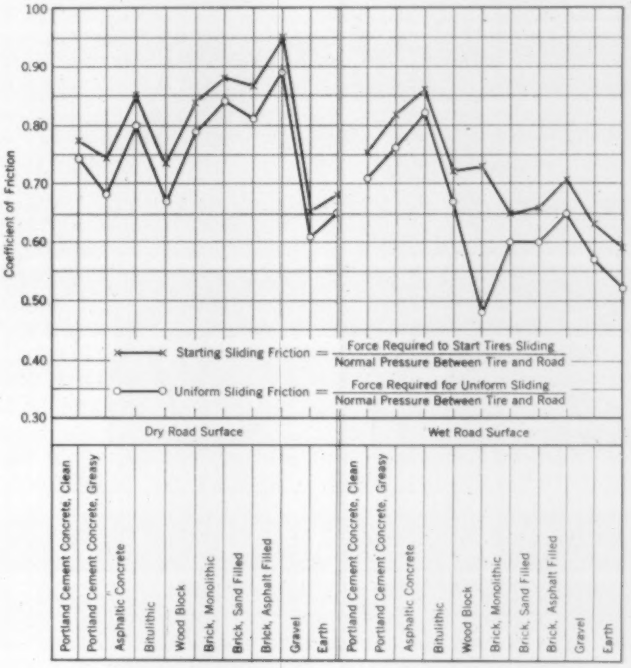


Fig. 2. Average Coefficients of Friction Between Tires and Road Surfaces

Each Value Is the Average of 40 Determinations with Four Wheels Sliding. Adapted from Bulletin No. 88 (1928), Engineering Experiment Station of Iowa

This would mean an amount of work varying between 1,200 ft-lb. (or 0.60 × 2,000) and 1,600 ft-lb. (or 0.80 × 2,000) per linear foot of distance through which the sliding takes place. If the kinetic energy of the vehicle amounts to 41,900 ft-lb. at the time the brakes are applied, the vehicle will slide a distance of between 26 ft. (or 41,900 ÷ 1,200) and 36 ft. (or 41,900 ÷ 1,600) before coming to a stop. In most instances, especially at the higher speeds, the brakes will not be applied hard enough to lock the wheels instantly, but will be used several times, causing successive impacts of a shoving effect as the wheels are partially locked and subsequently released.

Disturbance of the component parts of the surfacing, through starting and sliding friction, is detrimental to all types of roadway surfaces with the exception of the rigid types. In the case of the latter, the friction is absorbed by wear on the tires to a considerably greater extent than in the case of the non-rigid types.

The gross load of the vehicles using a highway is the predominant factor in the design of the surface, and therefore in its initial cost, the heavier loadings requiring a proportionally heavier design. The detailed analysis of this factor cannot be given in a limited space, but in general it may be said that a mathematical analysis indicates that the increase in stresses in the highway is in direct ratio to the vehicle loading.

OBTAINING HIGHWAY FUNDS

At present the trend of thought would indicate that funds for highway improvement and maintenance should be obtained entirely through a direct tax on the motor vehicles using the highway, and that no part of such funds should come from real and personal property. It is a question whether all forms of property do not increase in value to some extent through the proper development of a highway system and the material advantages it affords to the community or state as a whole. This enhancement of value is difficult to determine, but a small assessment on real estate would provide considerable revenue; it would be simple and inexpensive to collect, and would relieve the motor-vehicle owner of part of his burden.

The tax on the motor-vehicle owner or on the user of the highway should be equitably distributed among the various types of vehicles and proportioned according to the benefit they derive from, or damage they do to, the highway. An equitable distribution of such a tax is generally secured by a levy on the fuel consumed, commonly termed a gasoline tax, and by a fixed annual charge for the privilege of operating the vehicle, called a registration or license fee.

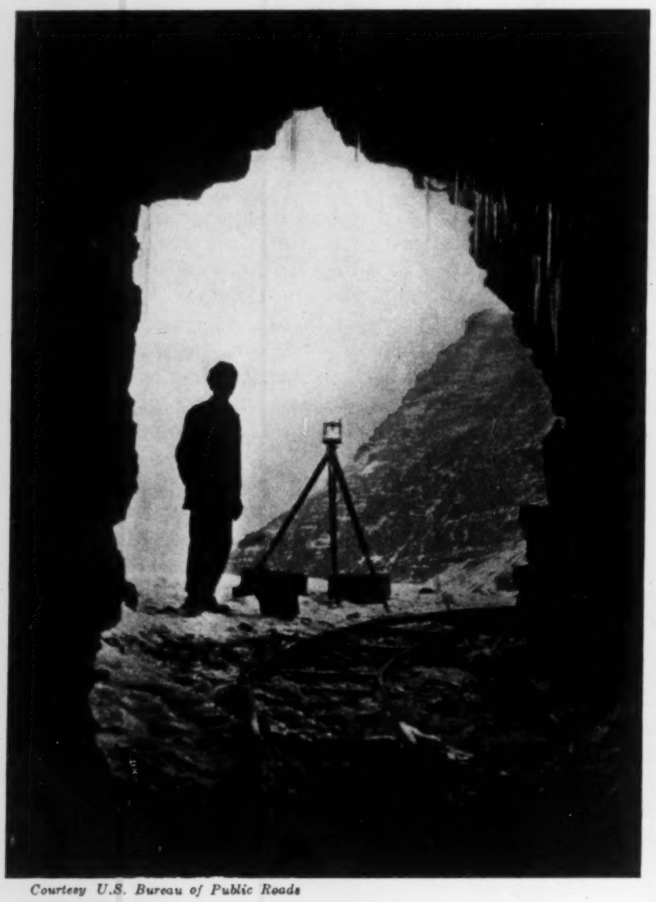
As motor vehicles consume a quantity of fuel in direct proportion to the mileage covered and the tonnage transported, the gasoline tax provides a ready yardstick for measuring the benefits derived, the light vehicle used intermittently incurring but a small charge, while the heavy vehicle used steadily assumes an increasingly larger charge. Therefore the fuel tax provides a fee primarily based on the amount of use to which the highway is subjected and should be regulated to provide revenue commensurate with the facilities afforded.

The registration fee assures the motor-vehicle owner the right to use the facilities of the public highway in transporting people or materials at his own convenience and is a charge for that privilege. Unlike the gasoline tax, the license fee is not conditioned on the extent for which the highway will be used, but on the facility that has been provided and is available. As the operating cost of vehicles depends to a considerable extent on the horsepower developed and the permissible loading, and as the damage done to the highway varies accordingly, it is logical to consider both of these factors in fixing the registration fee.

For example, assume that a 30-hp. vehicle of 2-ton gross load uses the highways to the amount of 15,000 miles annually and travels on the average 12 miles on each gallon of gasoline. The charges are to be based on a tax of $0.06 per gal. of gasoline used, and a license fee that includes horsepower at the rate of $0.10 per unit, and available loading at the rate of $0.01 per lb., as follows:

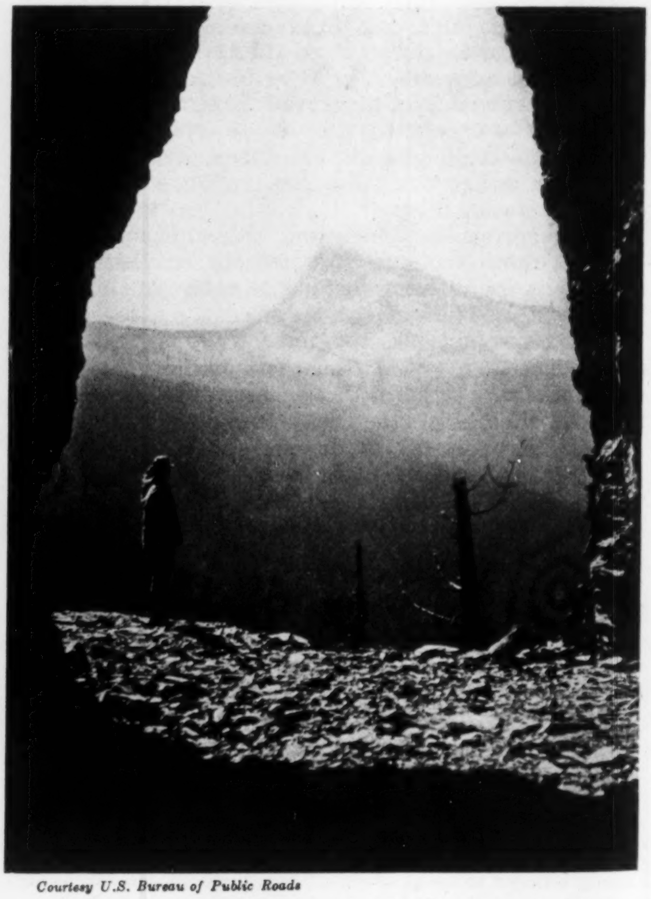
Registration fee:	
30 horsepower at $0.10	$ 3.00
4,000 lb. available loading at $0.01	40.00
Gasoline tax:	
1,250 gal. at $0.06	75.00
Total	$118.00

The total tax for the privilege of using improved highways in the instance cited would equal a monthly charge of about $10. If the operator realizes a saving of only $0.01 per mile by using improved rather than poor or mediocre highways, he saves 15,000 × $0.01, or $150 and in turn contributes $118 annually as his share of the cost of the improvement and its maintenance.



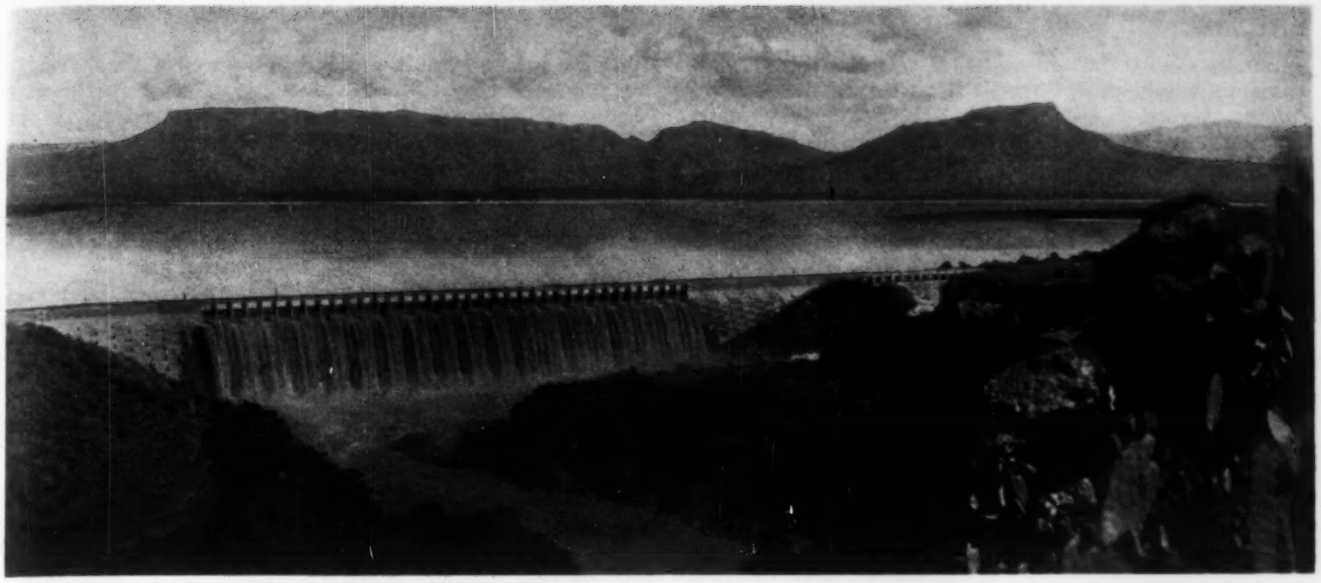
Courtesy U.S. Bureau of Public Roads

GLACIER NATIONAL PARK—PTARMIGAN WALL HIGHWAY TUNNEL
Looking Out Through the South Portal



Courtesy U.S. Bureau of Public Roads

VIEW THROUGH A WINDOW OF THE TUNNEL ON THE TRANS-MOUNTAIN HIGHWAY
Glacier National Park



Panorama of Van Ryneveld's Pass Dam Discharging 5,000 Sec-Ft.
Bar of Boulders Forming at the Entrance to the River

Erosion Below a South African Dam

Record Flood Safely Passed Over the Van Ryneveld's Pass Dam, a High Overflow Structure

By Philip R. R. Bisschop
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Resident Engineer, Van Ryneveld's Pass Irrigation District
Graaff Reinet, Union of South Africa

CONSIDERABLE attention has been given by the engineering profession to methods of protecting from erosion the channel below an overflow dam. Extensive investigation and research on this subject have been undertaken and in many instances have involved substantial capital expenditures. The results, here described, of the first spillway discharge on the unprotected toe of this high overflow dam will therefore prove of much interest, especially because the consequent erosion of the undisturbed topography below the dam was occasioned by a flood of unprecedented magnitude.

ON the Sundays River, immediately above the town of Graaff Reinet, in the Union of South Africa, is situated the Van Ryneveld's Pass Dam, built in 1924. A section of this overflow gravity structure is shown in Fig. 1. Its distinctive feature is the downstream face, which has been stepped over the full length and height. The length of the dam is 1,250 ft. and its crest is approximately 100 ft. above the original river bed. The reservoir has a capacity of 64,000 acre-ft.

The spillway section is made up of 52 openings, each with a clear span of 12 ft. 6 in. Of these, 37 provide a direct flow over the center part of the dam and the remaining 15 discharge into an auxiliary spillway excavated out of the rock of the east abutment. The channel thus excavated, which is roughly on a slope of one in ten, makes a right-angle bend and joins the main channel of the river at a point approximately 100 ft. below the downstream toe. Its discharge therefore runs at right angles to the discharge over the center section of the structure, as may be seen in the accompanying illustrations.

The dam was built across a canyon of the Sundays River, where a hard, fine-grained doleritic formation provided a sound rock foundation at approximately 30 ft. below the river bed. The surface topography of the canyon consists of two parts: the river bed proper, skirting the west side of the passage between the two abutting hills; and a level alluvial plain between 250 and 300 ft. wide occupying approximately three-quarters of the eastern side of the passage between the hills.

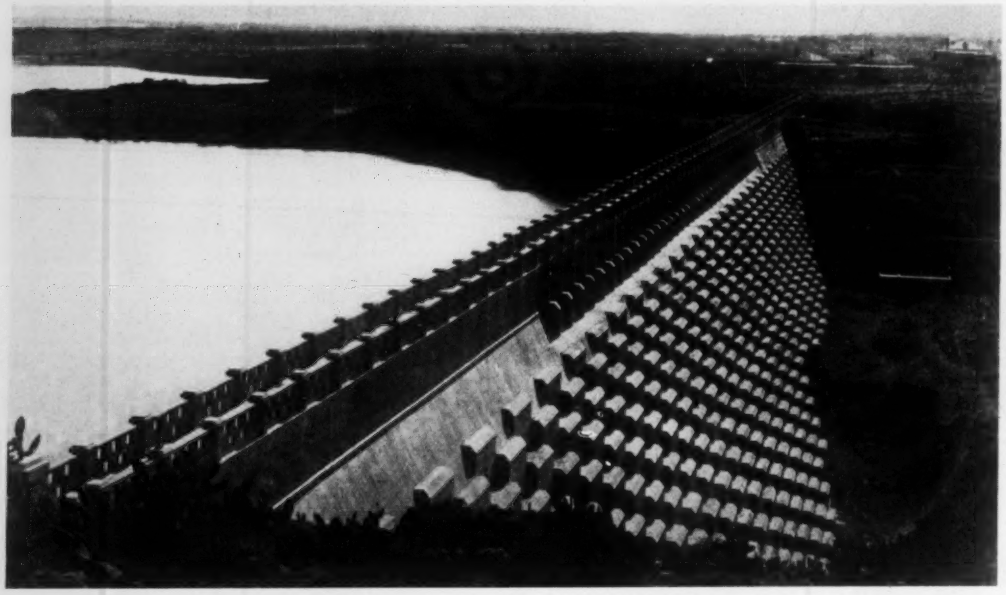
The ground surface of this plain is about 20 ft. above the original level of the river bed and consists of a top layer of sandy loam, comparatively free of boulders and approximately 15 ft. deep, overlying a thick layer of hard clay. This in turn overlies a stratum of river wash composed of boulders and gravel, which extends down to the doleritic rock. It is important to note that the discharge from the auxiliary spillway must cross this alluvial plain to reach the river channel proper, on the west side of the canyon.

At the time of construction no special provision was made for the development of a water cushion, as it was anticipated that a natural cushion would be formed by the scouring out of a certain amount of surface material, leaving a standing pool at the toe.

THE FLOOD AND CONSEQUENT SPILLWAY DISCHARGES

From December 29, 1931, to January 1, 1932, continuous rains fell over the entire southern coastal area of the Union of South Africa, culminating in record floods on New Year's Day. Along the coast, more than 24 in. of rain was recorded over considerable areas for that storm. Although the precipitation inland was markedly less, nevertheless the rainfall was considerably in excess of that recorded within the preceding fifty years. The normal annual rainfall is about 14 in.

On New Year's Day, 1932, it became clear that not only would the reservoir fill for the first time since its



THE STEPPED DOWNSTREAM FACE

Dam for South African Irrigation Project Stores 64,000 Acre-Ft. of Water

completion, but that in addition a flood of record dimensions would pass through it and over the spillway. Spilling started at 4:00 p.m. on New Year's Day, and at 1:00 a.m. on January 2 attained a maximum depth over the spillway crest of 5.10 ft., equivalent to approximately 26,500 sec-ft. The maximum flow into the reservoir lasted from approximately 7 to 10 p.m., January 1, at the rate of from 40,000 to 45,000 sec-ft., as against a previous maximum recorded discharge of the river of 31,000 sec-ft.

The behavior of the falling sheet of water on the overflow section is shown in the photographs. While the discharge flowed very smoothly over the ogee-shaped section of the spillway crest, the water shot out in a horizontal direction when it hit the topmost buttresses. When the depth of discharge became greater than about 12 in., the velocity induced in the ogee section was sufficient to make the water, after impinging on the first buttresses, jump clear of the rest of the concrete. The object of breaking the falling sheet of water into a spray by cascading it downwards from buttress to buttress was not wholly gained. A true cascade was only obtained when the depth of discharge fell below 12 in.

Directly adjacent and parallel to the downstream toe of the dam, the falling sheet of water, in dropping clear of the concrete, scoured out a well-defined trench in the hard clay formation from 12 to 15 ft. wide and about 10 ft. deep, or about 25 ft. below the original ground surface. The trench is boulder strewn to a considerable extent and includes the concrete foundation of some of the construction equipment.

In the auxiliary spillway the rock formation had been previously loosened and shattered by blasting, with the result that boulders of very considerable size, some at least 2 cu. yd. in content, were washed down by the raging torrent and deposited directly below the exit to the spillway as well as in the trench adjacent to the dam wall. Consequently the grade of the auxiliary spillway has been cut down in places more than 6 ft. to solid rock.

In the river section, the results of the flood were entirely different. Starting a short distance below the downstream toe of the dam and extending a few hundred feet downstream, a broad shallow bar of boulders was formed by deposition entirely across the river bed. The boulders forming the bar are of all sizes, the largest being about 12 cu. ft. in content. These were transported from the auxiliary spillway and the scoured part of the alluvial section. The water from the auxiliary spillway caused some scouring of the west river bank immediately downstream from the boulder bar as it changed its course through a right angle on entering the river channel. In addition, the river bed was somewhat widened, but the grade was not materially changed. Within 250 yd. below the dam the river resumed a normal flood flow, similar to the flows passing down the river before the dam was built, as is clearly illustrated in the panoramic view.

Comparatively few boulders are to be found in the strip on to which the water fell, adjacent to the downstream toe, below which the protecting bar was formed. As the overflow diminished, the trench gradually filled up with mud and muck from the clay formation brought down from the auxiliary spillway by the water. The present elevation of the boulder bar is about 3 ft. higher than the bed of the river was before the flood. It was the formation of this bar across the entire width of the river bed, at the exit of the

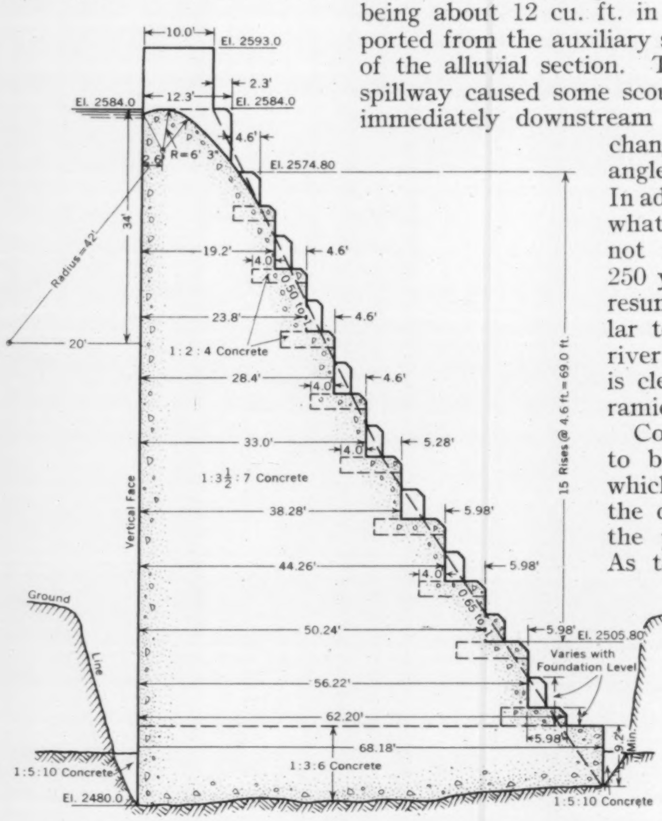
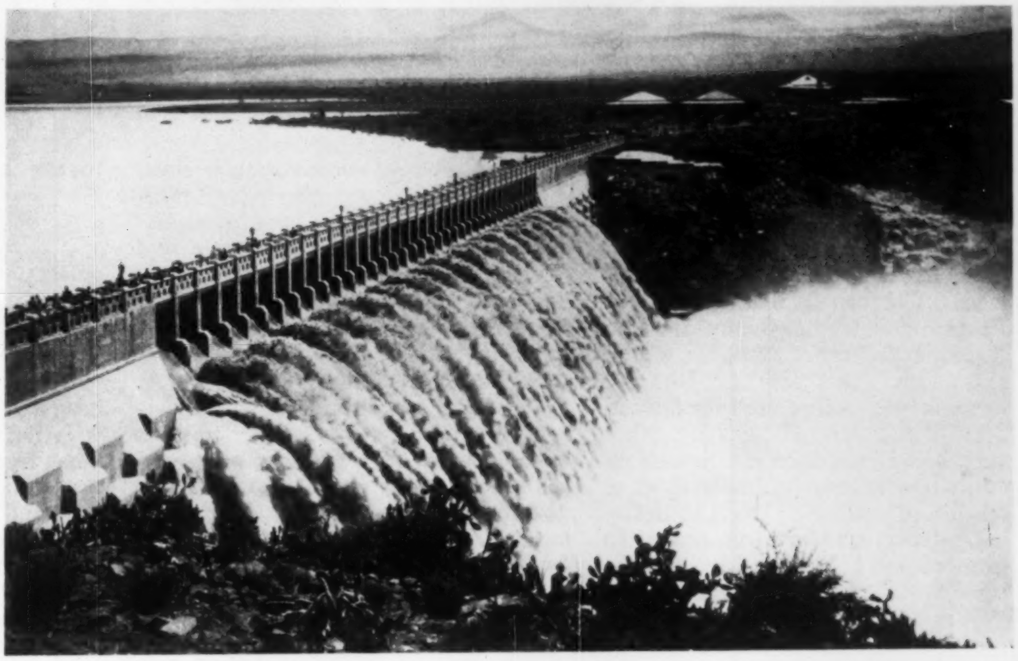


FIG. 1. CROSS SECTION OF THE VAN RYNEVELD'S PASS DAM, SUNDAYS RIVER

Straight Gravity Overflow Structure 100 Ft. High, Built in 1924 boiling cauldron, that effectively prevented any extensive scouring directly adjacent to the toe of the dam.

The formation of this boulder bar is the direct result of deposition and must have taken place at the same time that the soil and clay in the eastern section of the river canyon were scoured out. The larger boulders were probably deposited first, then the smaller material, tapering off gradually in size to muck and soil, which accumulated gradually and became firmly lodged against the boulders. For several weeks after the discharge from the spillway had ceased, a large pool of water remained at the upstream end of the bar, making it evident that the barrier is fairly water-tight.



A Fifty-Year Run-off Record Broken
20,000 Sec-Ft. Discharging from the Main and Auxiliary Spillways

DAM UNDAMAGED

Immediately after overflow had ceased, a thorough inspection was made of the entire area on the downstream side of the dam. Hardly any evidence of hydrostatic uplift pressure was apparent, and the dam appeared to be remarkably water-tight. The scouring had been surprisingly efficient; there was not the slightest need for any repair work.

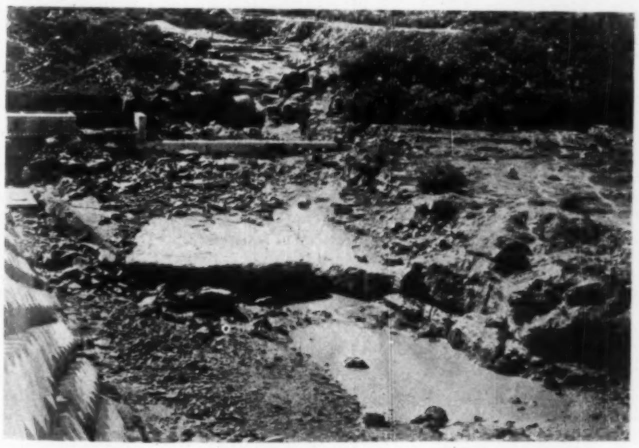
If the auxiliary spillway had not been constructed, the erosion would undoubtedly have been less extensive. On the other hand, without the shattered rock eroded from the spillway channel, it is doubtful whether the boulder bar would have formed as it did and whether it would have acted with the same efficiency.

Future discharges will probably erode the exposed clay formation across the exit of the auxiliary spillway, shown in one of the illustrations, until the concrete wall is met. As this wall was carried down to bedrock, the scouring action will not continue further. Apart from the hole that will naturally be formed immediately below the concrete wall, the general level of the channel will be determined by the present level of the boulder bar.

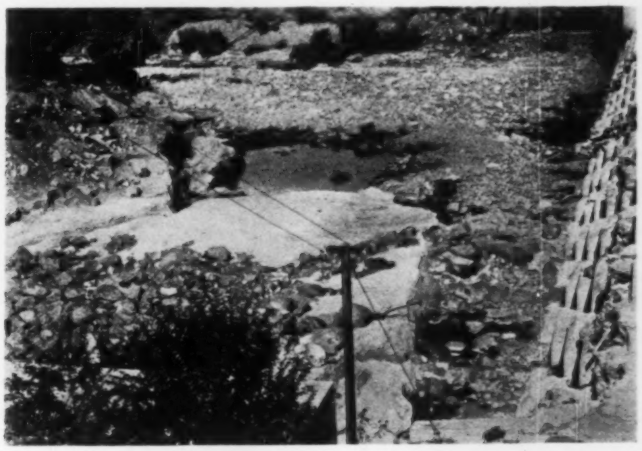
It is probable that the downstream bank of the auxiliary trench will be further undercut by the water as it swings around in entering the river channel proper. If there appears to be any danger that this undercutting will outflank the boulder bar, it may be necessary to build a few inexpensive training walls. So far as present conditions indicate, however, no further works will be required.

DESIGNED BY IRRIGATION DEPARTMENT

The Van Ryneveld's Pass Dam was constructed by the Van Ryneveld's Pass Irrigation District, according to the designs, and under the supervision, of the Irrigation Department of the Union of South Africa, of which A. D. Lewis is the Director. During the construction of the dam, K. R. Shand acted as Resident Engineer to the board of the irrigation district.



Erosion at Junction of Auxiliary and Main Spillways
Channel Eroded a Maximum Depth of 25 Ft. Below the Original Surface



Boulder Bar Formed Across the Spillway Channel
Note the Clay Formation in the Foreground and the Undrained Pool Behind the Bar

Deflection in Reinforced Concrete Beams

Simplified Formula for a Modern Theory Is Verified by Tests

By Donald M. Burmister
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Instructor in Civil Engineering, Columbia University, New York

In these days of conservation of time, energy, and materials and of standardization of methods and manufactured articles, engineers are more and more seeking to simplify processes. Deflection formulas for concrete beams have been developed and published elsewhere but in a form more difficult of application than that presented by Mr. Burmister in this article. The formula given here has been checked against actual tests in the laboratories at Columbia University and has been found to give results consistent with those obtained in practice, the agreement becoming closer and closer as the load on the beam increases.

Reinforced concrete beams and slabs should be designed in accordance with some requirements as to maximum permissible deflection under design load. This can be readily done for structural steel beams by the use of simple deflection formulas, giving reasonably exact results. But the formulas heretofore used for the deflection of concrete beams are either very complicated or are not in an easily usable form. The analysis of reinforced concrete beams to determine deflection should be similar to that of steel beams, because the same laws and assumptions are used in the flexural analysis of both. Reinforced concrete acts as a unit just as does a built-up girder and therefore is in a sense a homogeneous structural material. This assumption of homogeneity is substantiated by the fact that reinforced concrete can be designed with the reasonable certainty that there will be a close agreement between the results achieved by tests and those reached by computation.

This analysis is given to show that deflections likewise can be computed with sufficient accuracy. Starting with the simple deflection formula for steel beams, a relation has been developed for the deflection of concrete beams that is simple and of practical application. One of the most useful and most widely applicable formulas for steel beams expresses the deflection in terms of the stress on the extreme fiber and the depth of the beam in inches, as follows:

$$\Delta = \frac{C f L^2}{E d} \quad . \ . \ . \ . \ . \ . \ . \ . \quad [1]$$

in which L = span in inches
f = stress on extreme fiber
E = modulus of elasticity of steel
d = depth of beam to extreme fibers
C = a constant depending only on the method of loading the beam, in which

$$C = \frac{k_1}{k_2},$$

k_1 being the constant in the deflection formula:

$$\Delta = k_1 \frac{W L^3}{E I} \text{ and}$$

k_2 being the constant in the moment formula:

$$M = k_2 \, W \, L$$

This deflection formula states that for a beam of any given span, method of loading, and support, the deflection depends only on the extreme-fiber stress and the depth of the beam. The position of the neutral axis in a symmetrical steel beam is such that the distance, c, to the extreme fibers is equal to $\frac{d}{2}$. The deflection formula for steel beams can therefore be written as follows:

$$\Delta = \frac{C f L^2}{2 E c} \quad . \ . \ . \quad [2]$$

Applying this formula for steel beams to reinforced concrete beams, it is found that the deflection of a beam of any given span and loading depends only on the stress in the steel and the position of the neutral axis. The position of the neutral axis in a concrete beam is determined by the percentage of tensile reinforcement and by the value of n, the ratio of the moduli of elasticity of steel and concrete. The stress in the steel, for any given loading, is a function of the distance, c, of the steel from the neutral axis (Fig. 1).

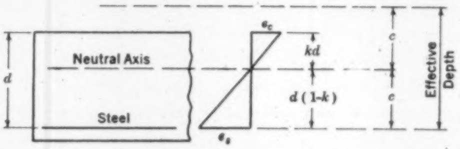


Fig. 1

The deflection of a concrete beam can therefore be expressed in terms of the fiber stress and the distance, $c = d(1-k)$, of the steel from the neutral axis, as follows:

$$\Delta = \frac{C f_s L^2}{E_s d \, [2(1-k)]} \quad . \ . \ . \ . \ . \ . \quad [3]$$

The effective depth of a reinforced concrete beam in resisting deflection is therefore equal to twice the distance of the steel from the neutral axis. That is, the stiffness of a concrete beam for a given fiber stress is greater than the actual depth would seem to indicate. For a balanced section, which is that commonly used, $k = 0.374$, and $2c = 2d(1-k) = 1.25d$. The effective depth is therefore 25 per cent greater than the actual depth, and the concrete beam is 25 per cent stiffer than the steel beam of the same depth and unit stress in the steel.

In Equation 3 we have a deflection formula that is very simple and practical in application. There is nothing radically new in this concept. In the paper entitled "Relations Between Deformations and Deflections in Reinforced Concrete," in the *Proceedings* of the American Society for Testing Materials, Vol. 14 (1914), G. A. Maney, M. Am. Soc. C.E., shows that the unit deformations in the steel and in the extreme fiber of the concrete are the only determining factors in the deflection for any beam. This relationship is verified by tests; a large number of plotted results show a very close agreement between the deflection curves obtained by tests and those plotted by computation. However, this formula is not readily usable in the form given.

because the strains in the steel and concrete are required for solution; that is:

$$\Delta = \frac{k\,L^2}{d}\,(e_s + e_c)$$

Starting with the stress-strain relationships and the necessary assumptions for flexure of beams (Fig. 2), and substituting in Formula 2:

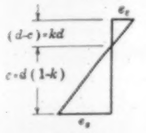


FIG 2

$$e_c : (d-c) = e_s : c$$

$$c = d\left(\frac{e_s}{e_s + e_c}\right)$$

Substituting in Formula 2,

$$\Delta = \frac{C\,f_s\,L^2}{2\,E_s\,d\left(\dfrac{e_s}{e_s + e_c}\right)} = \frac{C\,e_s\,L^2}{2\,d\left(\dfrac{e_s}{e_s + e_c}\right)} = \frac{C\,L^2}{2\,d}\,(e_s + e_c).$$

This is the form in which Maney's formula is given and shows that it may be reduced to the form of Formulas 2 and 3, which were derived from the steel-beam analogy.

The reasoning that applies to the steel beam has been carried over to the concrete beam. The effect of tension in concrete for low loads is to make the beam stiffer in the earlier stages of loading, and the deflections will consequently be less than those computed. The actual and computed deflection curves approach each other as the load on the beam is increased. Neglecting tension in the concrete, the deflection of beams under design load, that is, for the maximum allowable stresses in the materials, is however of more practical significance.

Values of the deflection constant C, for different methods of loading and supporting beams, are given in Table I. The significance of this table lies in the fact that for any given method of support, the distribution of the load makes comparatively little difference, as an inspection of the constants shows. For instance, for a simple span the constants vary only from 0.167 to 0.213 for the three types of loading.

TABLE I. VALUES OF C, DEFLECTION FORMULA CONSTANT

METHOD OF SUPPORT	METHOD OF LOADING	VALUES OF C
Cantilever	Concentrated load	$^2/_3$ = 0.67
	Uniform load	$^1/_2$ = 0.50
Simple span	Concentrated center load	$^1/_6$ = 0.167
	Third point loading	$^{23}/_{108}$ = 0.213
	Uniform loading	$^5/_{24}$ = 0.208
Fixed-end span	Concentrated center load	$^1/_{12}$ = 0.083
	Uniform load	$^1/_{16}$ = 0.063*

* For f_s at support.

A few values of $2(1-k)$ for different percentages of steel are given in Table II.

TABLE II. VALUES OF $2(1-k)$ FOR DIFFERENT VALUES OF THE STEEL RATIO, p

VALUES OF p	FOR n = 12	FOR n = 15
0.006	1.37	1.31
0.007	1.34	1.27
0.0077	....	Balanced 1.25
0.008	1.30	1.24
0.009	1.26	1.20
0.0094	Balanced 1.25	....
0.010	1.24	1.17

A practical application of the formula is to determine the minimum safe depth of beams and slabs for a given span and method of loading so that, for instance, the deflection shall not exceed $\frac{1}{360}$ of the span in inches under a load, with a fiber stress of 16,000 lb. per sq. in. The deflection relationship is as follows:

$$\Delta = \frac{L}{360} = \frac{16{,}000\,L^2\,C}{29{,}000{,}000\,d\;[2(1-k)]} \quad \text{or}$$

$$d = \frac{C\,L}{10(1-k)} \quad \ldots\ldots\ [4]$$

For beams and slabs of balanced section, Table III has been prepared. It gives the minimum safe depth of slabs and beams in terms of the span length in inches, so that the deflection shall not exceed $\frac{1}{360}$ of the span length, when the fiber stress is 16,000 lb. per sq. in.

TABLE III. MINIMUM SAFE DEPTH FOR BEAMS AND SLABS OF BALANCED SECTION SO THAT DEFLECTION SHALL NOT EXCEED $^1/_{360}$ OF THE SPAN

METHOD OF SUPPORT	METHOD OF LOADING	DEPTH OF BEAM IN TERMS OF THE SPAN IN INCHES
Cantilever	Concentrated	$\frac{L}{9.4} = 0.107L$
	Uniform	$\frac{L}{12.5} = 0.080L$
Simple span	Concentrated	$\frac{L}{37.5} = 0.0267L$
	Third Point	$\frac{L}{29.5} = 0.034L$
	Uniform	$\frac{L}{30} = 0.033L$
Fixed-end span	Concentrated	$\frac{L}{75} = 0.0133L$
	Uniform	$\frac{L}{100} = 0.010L$

The quantities given in Table III are the numerical coefficients of L in Formula 4, namely:

$$d = \frac{C}{10}\left(\frac{1}{1-k}\right)L$$

In this formula

C = the ratio $\frac{k_1}{k_2}$ obtained from Table I

k = the ratio of depth of neutral axis to effective depth d of the concrete

The value of k has been computed for f_s = 16,000 lb. per sq. in., f_c = 650 lb. per sq. in., and n = 15.

The approximate deflection for T-beams can also be found by this formula by taking the effective depth as equal to twice the depth of the steel below the flange. It can be more accurately determined by finding the exact position of the neutral axis. The stiffness of T-beams is very great compared to that of rectangular beams of the same depth because of the relatively high position of the neutral axis.

For example, consider that the problem is to find the minimum thickness of slab for a 10-ft. simple span, so that the allowable deflection will not be exceeded under the working load. The factor from Table III for a balanced section is $\frac{L}{30}$. The effective depth is therefore $\frac{10 \times 12}{30} = 4$ in. Consequently the deflection of beams does not control the design unless the span is very long and the beam very shallow. But if the amount of deflection under any given load is desired, it can be easily determined from Formula 3 with reasonable assurance of accuracy.

Engineering Uses for Geophysics

Sounding the Earth by Seismic, Magnetic, and Electrical Methods

By Sherwin F. Kelly

Low and Kelly, Consulting Mining Engineers, Geologists, and Geophysicists, New York

NEARLY every engineer is confronted at some time during his career with the problem of determining the nature and characteristics of the structure of the earth below the surface. Civil engineers, particularly, want to know how deep it is to bedrock and where the water-bearing formations are. In such cases drilling may be resorted to, but it involves considerable expense and often requires more time than is available.

Geophysical prospecting methods have been used for a number of years by mining and petroleum engineers. More recently they have been adapted by civil engineers for the preliminary exploration of dam sites, bridge foundations, tunnel location, and for the classification of highway excavation. Although many of the numerous methods available are described briefly by Mr. Kelly, he concludes that the electrical methods are the most useful and dependable for the civil engineer in his preliminary investigations.

THE earth is an engineering structure in the eyes of the geophysicist, and to its investigation he has been quietly applying engineering methods for a good many years. The actual instruments used may not always be familiar to engineers, but the aim is the same as theirs; to make a quantitative study of the properties of materials in order to predict their behavior under given circumstances. In order to find the densities and elasticities of materials deep within the earth, the geophysicist has made use of the earthquakes of the world. Seismic observations on the paths and speeds of earth-shaking tremors have told what materials make up the interior of our globe. Almost unbelievably delicate measurements on the variations of the gravitational, magnetic, and electrical fields of the earth have been made to yield information on the characters of hidden rock "structures" nearer the surface.

The engineer studies the properties of easily handled laboratory specimens, but the geophysicist has been forced to make his observations on a grand scale, often with inaccessible specimens, in the laboratory of the earth. The data thus obtained and the technic learned in the wide sweep of the earlier studies in recent years have been turned towards problems of smaller scale and more immediate economic interest. We have now arrived at the stage where the engineering study of the earth is made to serve the engineer, and the science and art of applied geophysics is assuming an important rôle in the solution of many practical engineering problems.

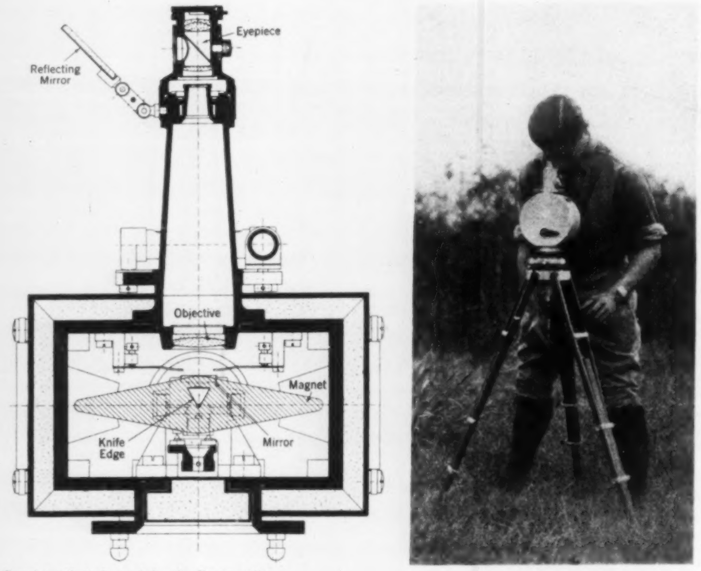
Quite naturally, the new geophysics was first applied to the search for valuable mineral deposits, with the result that the mining and the petroleum engineers were among the earliest to benefit by the new science.

With the independent researches of Conrad Schlumberger in France, Hans Lundberg in Sweden, and Harry Conklin and Frank Wenner in the United States, shortly before the World War, commercially practical methods were finally developed for applying electrical measurements to the solution of geological problems and to the search for ore. Their commercial use in the United States and Canada began when I introduced the Schlumberger methods in those countries in 1921.

Shortly after the introduction of electrical technics, seismic and gravitational methods were brought from Europe and applied to the oil fields on the coast of the Gulf of Mexico for the purpose of locating salt domes. These are great plugs of salt, a mile or so in diameter, thrust up through the overlying rocks. They are of commercial interest to the petroleum engineer because they are frequently accompanied by peripheral oil pools.

The success of geophysical exploration depends on the observation of the difference in certain physical characteristics between one formation of rocks or minerals and another. Seismic prospecting relies on a series of portable seismographs to record the earth vibrations that have been set in motion by exploding a charge of dynamite. The earth waves are reflected or refracted back to the surface by subsurface rock formations whose elasticities, and hence speeds of transmission of the waves, are greater than those of the overlying materials. These elastic waves in solid rock follow much the same laws of reflection and refraction as do waves of light in more transparent media.

Electrical methods in geophysics depend on observation of the differences in electrical resistivity shown by various rocks and soils. These two technics are related in that they impress an outside, artificial field of force on the area to be studied, and that this field can be varied in strength and magnitude to suit the problem under investigation. In this they possess an advantage over the gravitational and magnetic methods, which are confined to the study of small variations in the



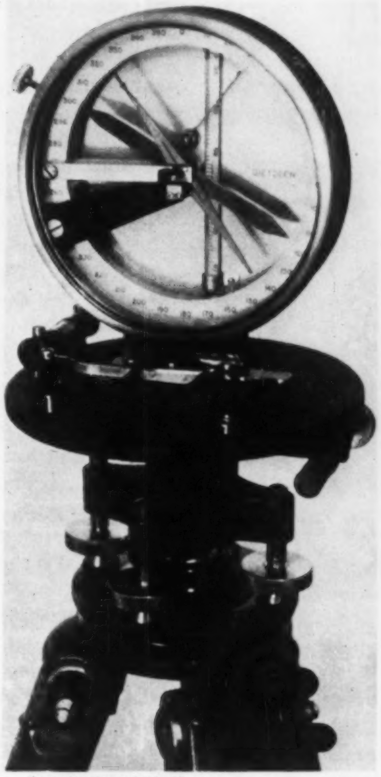
Courtesy American Askania Corporation

Fig. 1. The Schmidt Magnetic Balance
At Left, Section Through a Vertical Balance;
At Right, Newest Type of Horizontal Balance

natural fields of force proper to the earth itself.

Gravitational exploration utilizes a special torsion balance to measure the warp produced in the field of gravity by masses of rock of greater or less density than their surroundings. In magnetic work, small variations in the direction and intensity of the earth's magnetic field are recorded, these being due to the different magnetic effects of various underlying rock formations.

The scope of the magnetic, seismic, and gravitational methods in engineering is still largely conjectural, except in the field of economic geology and mining engineering, but civil engineers have already found the electrical ones to be of value in several types of work. Nevertheless, the magnetic and seismic technics deserve some mention before proceeding to discuss the electrical ones, if only to stimulate inquiry along these lines.



Courtesy W. C. McBride, Inc.

A Modified Sensitive Dip-Needle
The Hotchkiss Superdip

USING MAGNETIC METHODS

The lines of magnetic force of the earth's field are affected, both in strength and direction, by the magnetic properties of the near-surface rocks through which they pass. The effect will depend on the magnetic permeability, or "conductivity," and the sizes of these rock masses. Practically all the commonly encountered rocks have weak permeabilities, which differ in the different rock types. In the order of decreasing permeability they are: basalt, gabbro, granite, average sedimentary rock, and limestone. These differ so slightly in this property, however, that it is only when they are present in large masses that a measurable effect can be observed as between one formation and another. With adequate geological control and sufficiently delicate measurements, magnetic methods can be of service when the solution of larger-scale geological problems is of interest to the engineer. They may be of aid when it becomes advisable to map the concealed geological formations through which a tunnel is to be driven, or when, in the search for water supplies, the discovery of the position and condition of a known or suspected water-bearing formation depends on learning the character of the subsurface geology.

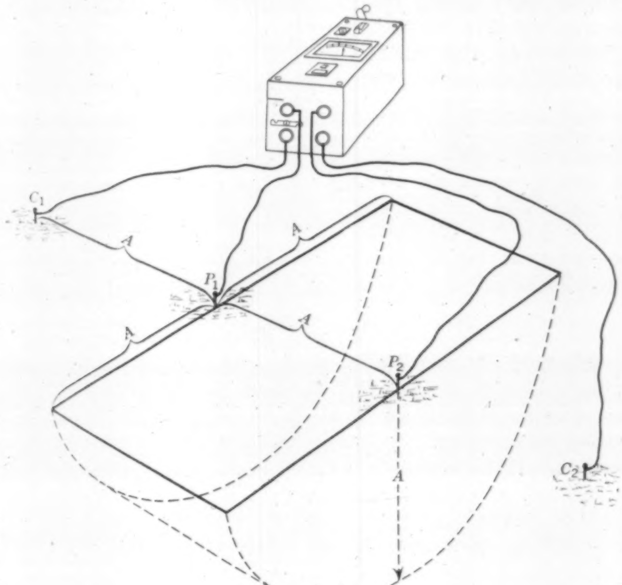


Fig. 2. Electrical Exploration Explained
The Volume of Ground Affecting the Measurements When the Stakes Are Equally Spaced at a Distance *A*

The usual practice is to measure variations in the horizontal or vertical component of the earth's field with a Schmidt magnetic balance, or to detect the changes in the inclination and total intensity with a magnetic dip-needle. The vertical magnetic field balance consists of a magnetic needle swinging in a vertical plane, on quartz knife edges, as shown in Fig. 1, with appropriate scales and an optical magnifying system to permit close reading of extremely small deviations from the zero or horizontal position. When set to read zero at an arbitrarily selected datum station, any variations from this position at other stations indicate changes in the vertical pull on the needle, that is, the variation in the vertical component of the magnetic field at each of those points. Similarly, the horizontal balance, a new type of which is illustrated, measures the variations in the horizontal component by means of a magnet system that normally stands vertically.

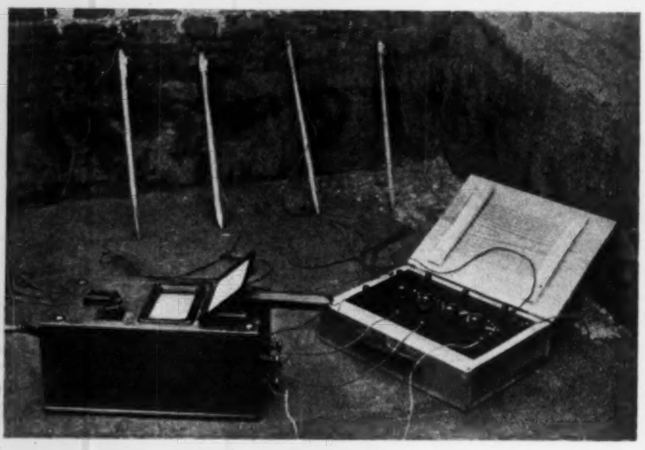
The dip-needle, long used to locate magnetic iron ores, consists of a compass needle mounted to swing freely in the vertical, instead of the horizontal plane, thereby aligning itself in the direction of the magnetic lines of force and indicating changes in their inclination. In the Hotchkiss Superdip, an auxiliary arm is added whereby the magnetic moment and the moment of gravity can be so balanced as to give the instrument greater sensitivity.

AN AID IN LOCATING WATER

These magnetic instruments have been used with success, for example, by the State Geological Survey of Missouri to aid in determining the subsurface geological formations for the purpose of finding water. A series of magnetic measurements along east-west lines revealed strong "magnetic highs" at various places. These were found to correspond with ridges in the granite and "porphyry" rocks composing the igneous "basement" formation, which underlies the sedimentary formations of Missouri at depths of from 100 to 2,000 ft. In places these ridges project through to the surface, but all known ridges, visible or buried, produce magnetic highs. This is because the igneous rocks are magnetically more permeable than the surrounding sedimentary rocks, into which they protrude locally.

In itself, this would have little to do with water supplies were it not for the fact that the sedimentary rocks have been locally fractured along the flanks of these ridges by minor geological disturbances. These fractured places now provide a better flow of water than other sections of the same rocks. Therefore, when a new magnetic high was mapped, a hitherto unknown, buried ridge was assumed to exist, and by experience it was established that water wells located on the flanks of these highs yielded a better flow than those in other localities, because they were in the fractured zone.

Magnetic highs do not always indicate promising sources of water. Far from it! Some geological control must be available before the magnetic readings can be interpreted. To achieve a worthwhile survey, the geophysicist must be a capable geologist, or have the wholehearted cooperation of one.



A MEGGER GROUND TESTER

Attached Through a Switchboard, at Right, to Its Ground Stakes

Seismic methods can also be used to study subsurface geological structures, but are rather expensive except when used to cover very large areas. An inexpensive way of treating small-scale problems is provided, however, by a modified seismic process employing a geophone to receive signals generated, for example, by the blow of a sledge on an anvil. The geophone amplifies sound waves transmitted through rocks on much the same principle as that upon which the doctor's stethoscope operates, and was used during the war to detect enemy sapper activities. It is employed in mine rescue work to locate entombed miners; it is used to locate knocks and faulty working parts in engines, and the suggestion has been made that it could serve to locate leaks in water mains by detecting the sound of the escaping water. With two geophones properly disposed, the direction from which sound waves are coming can be determined, which provides a means of measuring the depth to bedrock by observing the sound waves reflected from the buried rock back to the surface of the ground.

CALCULATING DEPTH TO BEDROCK BY ELECTRICAL METHODS

The knowledge of the depth of soil that covers bedrock is useful to the highway and railroad engineer who must calculate the cost of excavating rock and soil in cuts along the right-of-way. To the bridge or dam engineer, a sure knowledge of bedrock contours is a prerequisite to the design of his foundations.

The geophysical method most extensively used for determining the depth to bedrock is the electrical one. In the last three or four years, some thirty or forty dam sites in the United States, Canada, and Europe have been successfully studied by one electrical technic or another. The State Highway Department of Missouri is using a simple instrument for obtaining the depth of soil in proposed cuts, and the Illinois Geological Survey employs the same type of instrument for studying some of its geological problems. Incidentally it has found the technic useful in locating water supplies. The electrical methods seem better adapted than the others for all-round utility, and except for special problems they are the most useful and dependable of the geophysical technics for the civil engineer.

Fundamentally, electrical methods of geophysical exploration depend on the fact that different types of rock possess measurably different electrical resistivities. These are not due to inherent variations in the conductivity of the component minerals, because practically all rock-forming minerals are non-conductors, but to the fact that all rocks contain a certain amount of pore moisture, which is electrolytically conductive because of its content of dissolved, dissociated salts. Although these characteristics vary to some extent within each rock type, they do nevertheless remain broadly characteristic and definitive for each main group of rocks. Thus, granite and igneous rocks are drier and more compact than sedimentary formations and hence they are more resistant. Similarly, the various types of sedimentary rocks differ among themselves, shales being more conductive than sandstones or limestones.

Such measurements can most easily be made with the technic devised by Frank Wenner, of the U.S. Bureau of Standards. A current of, say, 50 volts is passed through the ground between two metallic stakes driven into the earth, and its amperage is measured. At the same time the drop in potential is observed across two intermediate stakes, driven into the ground between the first two. Then this voltage drop divided by the measured amperage gives the resistance in ohms between the two inside stakes. The field set-up is shown in diagrammatic form in Fig. 2. Since the resistance will vary with the distance separating the stakes, it becomes necessary to calculate the resistance of some unit volume of the ground, that is, the resistivity of the terrain involved in the measurement, in order to secure comparable data. A simple formula expresses the relationship of resistance to resistivity when the four stakes are in a straight line and at equal distances apart:

$$r = 2\pi A\ R \quad . . [1]$$

in which R is the resistance in ohms between the inside stakes, obtained as previously explained; A, the distance in meters between one stake and the next; and r, the average resistivity in meter-ohms. The resistivity in this case is calculated as the ohms-resistance of a cube one meter on an edge. Other units, such as foot-ohms or centimeter-ohms, could be used simply by measuring the distance between the stakes in feet or centimeters.

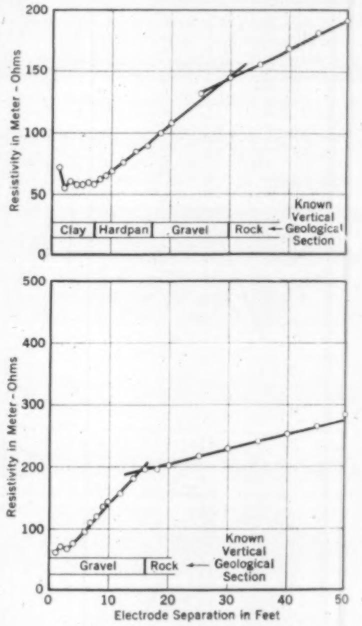


FIG. 3. RESULTS OF TWO BRIDGE-SITE SURVEYS

Breaks in the Curve of Relation Between Depth and Resistivity Indicate Changes in Geologic Formation

With the arrangement previously described, and shown in Fig. 2, it is possible to obtain the average resistivity of the volume of ground bounded by planes passed through the two inside stakes perpendicular to the line joining them, and contained within a radius, *A*, of that line. The depth of the ground investigated is therefore equal to the distance between the stakes, or the "electrode" separation. This fact evidently provides the experimenter with a means of controlling the depth to which observations are made; the deeper the phenomenon to be studied, the greater must be the electrode separation. By starting with a small distance between stakes and gradually increasing it, successive measurements will apply to deeper and deeper slices of ground. An abrupt change in the values obtained for average resistivity at a given electrode separation indicates that at the corresponding depth the current is penetrating material of a resistivity distinctly different from that which overlies it. Since porous, moist soil is normally a better conductor than compact, drier rock, the break in the resistivity curve is often quite marked at the electrode separation, corresponding to the depth at which the transition occurs.

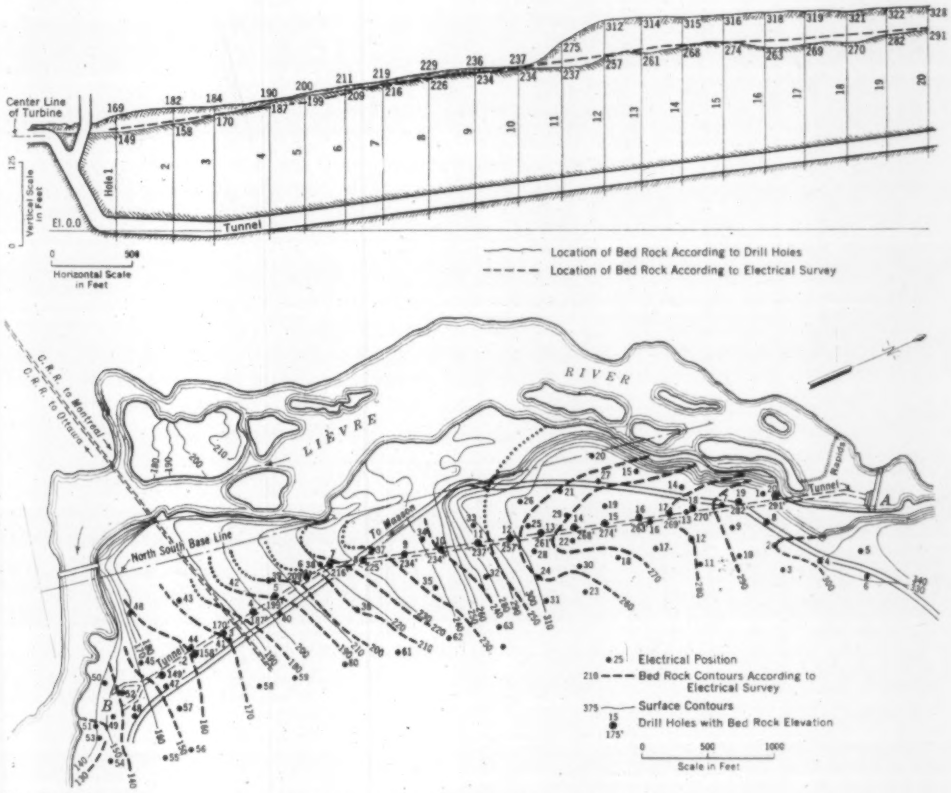
An example of the application of this procedure is given in Fig. 3, where the average resistivities are plotted as ordinates against the corresponding electrode separations, or depth, as abscissae. The succession of underlying formations is also shown to scale along the abscissae axis, bringing out the fact that the curve shows a break at the electrode separation equal to the depth to bedrock. Instances are sometimes encountered in which the curves are smooth, when other more complicated methods of interpretation are resorted to in order to arrive at depth determinations. This technic of determining the depth of overburden is quick and cheap and of evident applicability where foundation conditions are to be studied at proposed bridge or dam sites. It provides a rapid means of investigating several alternative locations in order to choose the most advantageous, and when a site has been decided upon, it can be used in conjunction with drilling for the detailed mapping of subsurface rock contours. A series of such measurements along proposed railroad or highway cuts will quickly indicate the relative amounts of rock and soil excavation that will be required.

Exploring Bedrock at Bridge Site; Megger Ground Tester and Gish-Rooney Instrument Checking Each Other

The result of a survey of the depth of cover along a proposed tunnel line is shown in Fig. 4. This work was carried out by Schlumberger methods, which differ only in detail from that already described. The bedrock contours are given, as for a dam site survey, and also a cross section along the tunnel line, as afterward checked by the drills used to test the quality of the bedrock itself. Some of the apparent discrepancies are due to the fact that the electrical determination and the drill core were not always taken at precisely the same spot.

Instead of investigating successively greater depths at each of a large number of stations, the variations of resistivity at a given depth over a large area may be studied. This is accomplished by keeping the electrode separation constant at a figure corresponding to the desired depth of investigation, and moving the whole configuration to successive stations located along the prolongation of the line joining the electrodes. With a series of such lines paralleling each other, a large area may be quickly and thoroughly explored.

In an experimental survey at the foundation for the Sarrans Dam now under construction in France, sound granite bedrock showed a resistivity of over 5,000 meter-ohms, whereas decomposed, broken, and permeable granite showed resistivities of about 3,000 meter-ohms. There is a definite relation, then, between resistivity and the degree of alteration of certain foundation material, sufficiently pronounced to forecast the suitability of some rocks for the purposes required.



After E. G. Leonardon

Fig. 4. Electrical Exploration for a Power Tunnel

Bedrock Contours Predicted by Electrical Methods and Checked by Drilling

In constructing a tunnel on the Bridge River power development in British Columbia, a highly shattered and water-soaked formation was unexpectedly penetrated, requiring a bulkhead at some distance from the heading of the partially completed tunnel. An electrical survey traced the direction of the water-bearing formation, which was highly conductive. An offset tunnel was constructed approximately parallel to the original alignment; and this time the contractor, being forewarned where to expect the troublesome formation, took the necessary precautions and passed through the danger zone safely. In these two surveys Schlumberger methods were used.

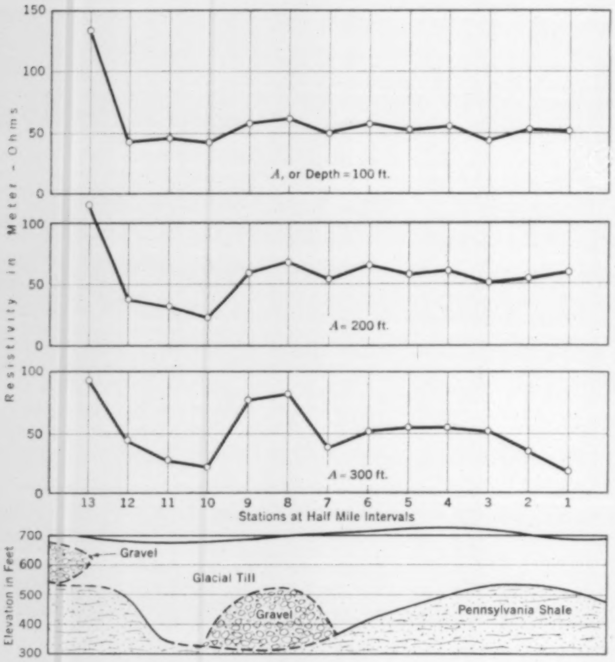


FIG. 5. RESISTIVITY SURVEY LOCATES WATER-BEARING GRAVEL
By M. K. Hubbert, Illinois Geological Survey

Results of an electrical investigation by M. K. Hubbert, of the Illinois Geological Survey, are plotted in Fig. 5. In this case, only one line of measures was followed, but successively deeper "slices" were studied by three series of measurements with three different electrode separations. From conclusive experiments nearby, where known bodies of water-bearing gravel in the glacial soil gave identical reactions, such a body of gravel was postulated here. The high resistivity recorded is due to the fact that the contained water is fairly free of dissolved salts and is therefore a poor conductor.

In carrying out this work, the Illinois Geological Survey employed a Megger ground tester, shown in the accompanying illustrations. This is an instrument long used by power and telegraph and telephone companies to test the resistance of ground connections to the earth, and which has recently been found adaptable to geophysical explorations. It consists of a hand-driven generator, giving about 50 volts, connected to the current element of an ohm-meter, thence to a commutator where it is converted to about 50-cycle alternating current, and then to binding posts connecting with the two outside stakes. The two inside stakes are connected to a commutator, converting the alternating current back to direct current, whence it passes through the potential element of the ohm-meter, thus indicating on it resistance in ohms between the two inside stakes.

The Missouri Highway Department is using a Megger ground tester in its estimation of rock and soil excavations. This tester gave me satisfactory results at the two bridge-site tests already mentioned and has also been found useful in corrosion studies. Exhaustive tests with various instruments carried out by the U.S. Bureau of Standards indicate that the electrical characteristics and the corrosiveness of soils are often related. The electrical conductivity of alkaline soils is a rough index of their corrosiveness, and where their resistivity falls below 5 or 6 meter-ohms, destructive corrosion can be expected. Higher values of resistivity may indicate corrosive areas when the figures are relatively low with respect to the surrounding soils. The relation between corrosion and electrical conductivity is not so definite in the case of acid soils, but even then the spots of especially low resistivity can be expected to be so-called "hot-spots" of corrosion.

Electrical soil surveys along the proposed routes of water, gas, and oil pipe lines, coupled with occasional chemical analyses, would often be valuable in determining the points at which special precautions should be taken against corrosion. Even in the case of smaller structures, where there is less contact between metal and ground, such studies might prove profitable. Electrical conductivity studies on a line already in place have been found to yield information as to where corrosion is taking place and, in the case of coated pipe, to indicate whether the coating is still effective or not. This method can also be used in locating buried oil and water lines, the exact locations of which are not known.

Before leaving the subject of electrical exploration, one more example should be cited, this time concerning its application in the home. Mr. Bela Low, my partner, has a summer place near New York, in the grounds of which a septic tank was buried a few years ago. The marker had been lost, and it was desired to locate this tank, which was known to lie within an area some 20 ft. on a side. Readings were taken with a Megger tester every 7 ft. along Lines 1, 2, and 3, 10 ft. apart. The electrode separation was 7 ft. An area of low resistance was found, but a test with a crowbar thrust into the ground quickly revealed that this was an abandoned cesspool, filled with mud. Evidently the mud was responsible for the good conductivity at this point. Intermediate Lines 4 and 5 were then run. A still lower reading was obtained at one point, and a hole was dug there. At a depth of about 2 ft. the top of the metal tank was encountered.

In presenting this material on geophysics, the aim has been to give a hasty perspective of some parts of its field of application, rather than to discuss the fundamental theories and instrumental technics. To lay any such theoretical groundwork would require several articles longer than this one, and they would probably be to little purpose. After all, the civil engineer is more interested in how geophysics can help him than in how it works. The final decision of whether to use geophysics in his work, and how to use it, he will have to leave to the expert in that line. Yet some understanding of its principles is needed to arrive at a clear comprehension of its possible utility. I hope that this brief explanation of some of these principles, coupled with the few examples of their application, will show civil engineers that geophysics can be of real service to them in many of their problems. Possibly it will stimulate thought along these lines, with the result that other types of problems will be revealed in which geophysics can save them time, money, and trouble.

Reconstructing the Sandbar Ferry Bridge

Sixty-Year-Old Structure of the Charleston and Western Carolina Railway Company Strengthened, Raised, and Lengthened

By J. E. Willoughby

Member American Society of Civil Engineers
Chief Engineer, Atlantic Coast Line Railroad Company, Wilmington, N.C.

INCREASING loads on the single-track line leading into Augusta, Ga., demanded the strengthening of the steel through-truss bridge carrying the Charleston and Western Carolina Railway over the Savannah River. In 1929 a flood of unprecedented proportions in the Savannah Valley swept out some of the bents of the approach trestle, scoured the river bed, and cut a new channel. Reconstruction became an immediate necessity to avoid future interruptions to traffic. To make maximum use of the original construction, the old brick piers were strengthened with envelopes of reinforced concrete supported on peripheral rows of foundation piles. Wooden approach trestles were replaced by concrete-pile and precast-slab construction. By raising the track 4 ft. and lengthening the bridge slightly, it was made possible safely to pass a flood of half a million second-feet, which is one-third greater than any known flood. Many ingenious construction methods were necessary to maintain traffic during construction.

NORTH CAROLINA
Spartanburg
Greenville
Anderson
SOUTH CAROLINA
Sand Bar Ferry
Augusta
C. & W. C. Ry.
GEORGIA
Atlantic Ocean
Port Royal

FROM Port Royal, S.C., the Charleston and Western Carolina Railway extends to Spartanburg, S.C., via Augusta, Ga., a distance of 246 miles, not including the branches to Anderson and Greenville, S.C. It is a part of the Atlantic Coast Line system of railways, and the Atlantic Coast Line trains into and out of Augusta are operated over its tracks between Augusta and Robbins, S.C. In passing through Augusta the railroad crosses the Savannah River twice—once at Sandbar Ferry, below the city, and again 12 miles above the city.

During the great flood in the Savannah River Valley of September and October 1929, when the discharge of the river at Augusta reached a maximum of 383,000 sec-ft., all the highway and railway bridges across the river at Augusta and between it and the sea were overtopped or damaged. At the Sandbar Ferry Bridge of the Charleston and Western Carolina Railway, a new overflow channel was dug out near the west end of the wooden approach trestle, and considerable scour took place in the channel of the river and around the brick piers. The wooden trestle was broken by drift and scour.

To prevent future interruption of train service by flood and to provide for greater axle loads, the railroad management decided early in 1931 to reconstruct the bridge. Although the 1929 flood was the greatest ever known—including the traditional Yazoo flood of 1796—the engineers decided that prudence required provision for a possible flood discharge of 500,000 sec-ft. This greater discharge has been allowed for by lengthening the bridge, by raising its floor, by replacing the wooden trestle approaches with steel and reinforced-concrete slab spans, and by re-forming to some extent the section

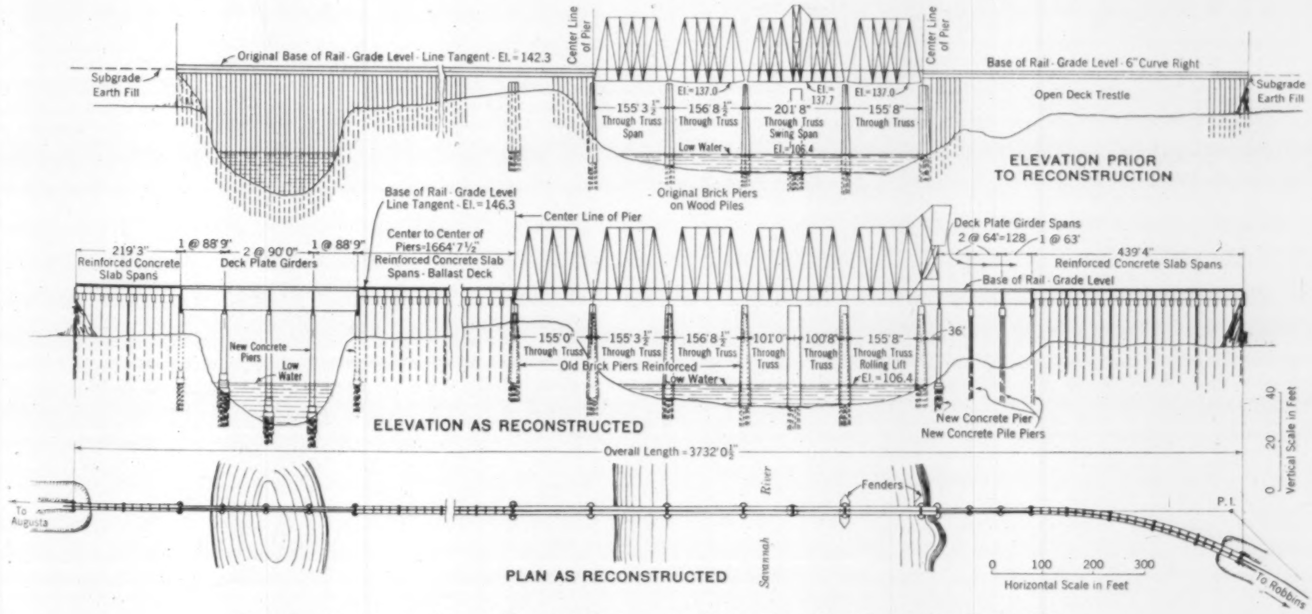

Center Line of Pier
Original Base of Rail - Grade Level - Line Tangent - El. = 142.3
Subgrade Earth Fill
El. = 137.0
155' 3 1/2" Through Truss Span
156' 8 1/2" Through Truss
201' 8" Through Truss Swing Span
El. = 137.7
El. = 137.0
155' 8" Through Truss
Low Water
El. = 106.4
Center Line of Pier
Base of Rail - Grade Level - 6° Curve Right
Open Deck Trestle
Subgrade Earth Fill
ELEVATION PRIOR TO RECONSTRUCTION
Original Brick Piers on Wood Piles
Base of Rail - Grade Level Line Tangent - El. = 146.3
Center Line of Pier
219' 3" Reinforced Concrete Slab Spans
1 @ 88' 9"
2 @ 90' 0" Deck Plate Girders
1 @ 88' 9"
Center to Center of Piers=1664' 7 1/2"
Reinforced Concrete Slab Spans - Ballast Deck
Deck Plate Girder Spans 2 @ 64'=128 - 1 @ 63'
439' 4" Reinforced Concrete Slab Spans
Base of Rail - Grade Level
New Concrete Piers
Low Water
155' 0" Through Truss
155' 3 1/2" Through Truss
156' 8 1/2" Through Truss
101' 0" Through Truss
100' 8" Through Truss
155' 8" Through Truss Rolling Lift
36'
Old Brick Piers Reinforced
Low Water
El. = 106.4
New Concrete Pier
New Concrete Pile Piers
ELEVATION AS RECONSTRUCTED
Overall Length = 3732' 0 1/2"
0 20 40 Vertical Scale in Feet
To Augusta
Savannah River
Fenders
P. I.
PLAN AS RECONSTRUCTED
0 100 200 300 Horizontal Scale in Feet
To Robbins

FIG. 1. SANDBAR FERRY BRIDGE OVER THE SAVANNAH RIVER AT AUGUSTA, GA.
Before and After Reconstruction in 1931 by the Charleston and Western Carolina Railway Company

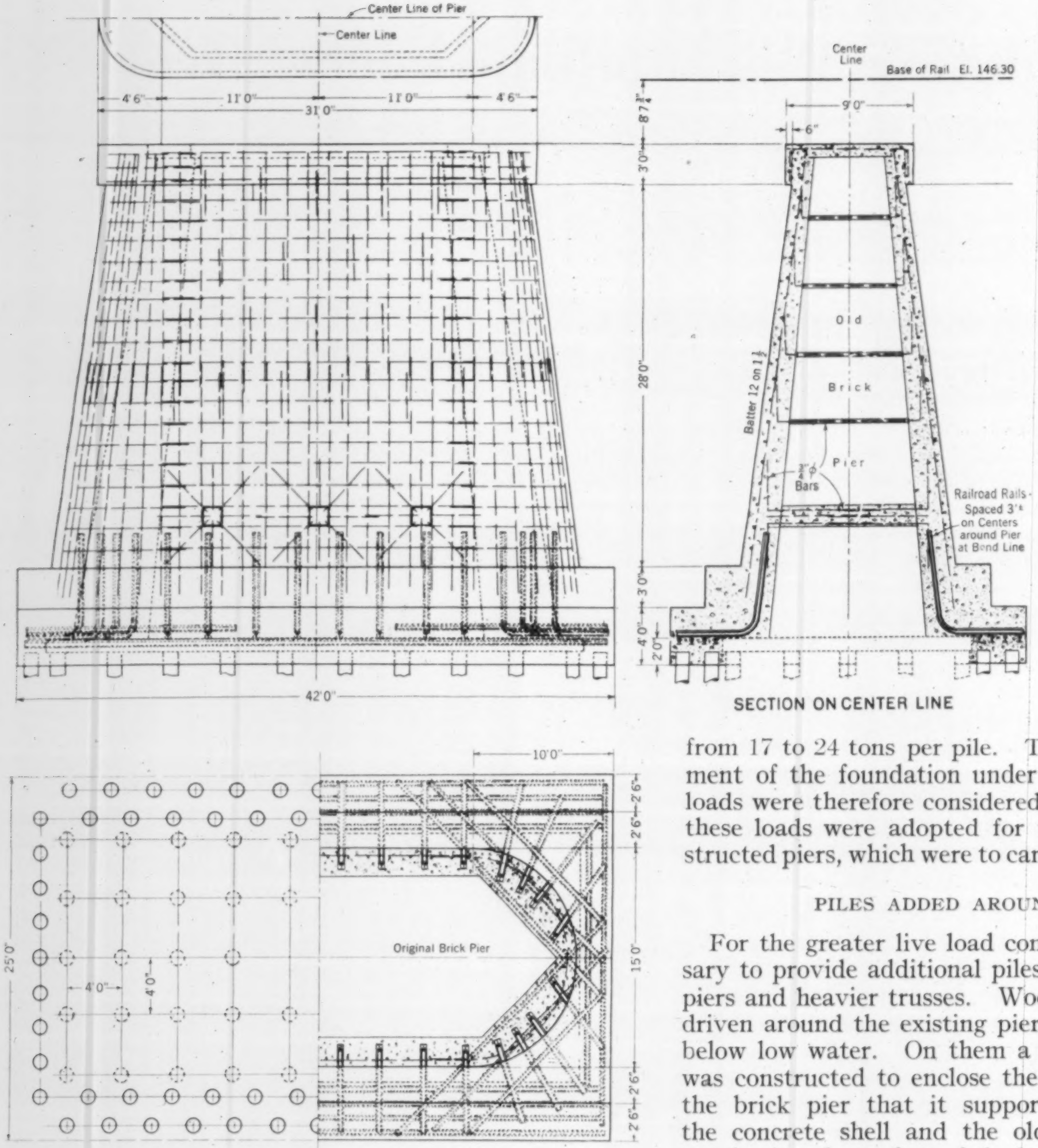


Fig. 2. Method of Strengthening the Old Brick Piers
Additional Piles Driven and Pier Encased with Reinforced Concrete

of the river channel and reducing its roughness coefficient.

As first constructed in 1872 and 1873, the bridge across the channel consisted of four wooden through-truss fixed spans, each approximately 155 ft. long, and one wooden through-truss draw span 200 ft. long. The flood plain was crossed with a wooden open-deck pile trestle. After one of the wooden trusses failed under a train, all except two of the wooden truss spans were replaced in 1894 by iron through-truss spans, and the westernmost of the wooden trusses was replaced by an open-deck pile trestle. The carrying capacity of the iron trusses was approximately equal to that of a bridge designed in accordance with Cooper's E-25 loading. In 1903 the trestle just west of the iron spans was replaced for a length of 155 ft. by a steel through truss designed for a live load equivalent to Cooper's E-40 specifications. In no case was the substructure strengthened when the trusses were replaced. The structure as it was before 1931 and as reconstructed at that time is shown in Fig. 1.

The original substructure was built of brick laid in cement mortar on a grillage of two thicknesses of 12 by 12-in. Georgia pine, which was in turn supported by wooden piles. The grillage was placed below extreme low water. In 1931 the masonry was in excellent condition, as was also a part of the timber grillage of one of the piers that was cut away at that time. The calculated load on the pile supports of the piers at the time of reconstruction, under the locomotive live loads that then obtained, varied from 17 to 24 tons per pile. There had been no settlement of the foundation under these loads. These pile loads were therefore considered safe, and approximately these loads were adopted for the design of the reconstructed piers, which were to carry the higher live loading.

PILES ADDED AROUND OLD PIERS

For the greater live load contemplated, it was necessary to provide additional piles to support the enlarged piers and heavier trusses. Wooden piles were therefore driven around the existing pier foundations and cut off below low water. On them a reinforced concrete shell was constructed to enclose the original foundation and the brick pier that it supported. Adhesion between the concrete shell and the old pier was obtained by roughening the surface of the brickwork. The two sides of the concrete shell were tied together by drilling holes through the brickwork and inserting ties of steel rods. It was assumed that the old and new foundations would act as a unit.

In the foundations of the reconstructed bridge, the load on each pile will vary from 16 to 22 tons. A typical pier, as reconstructed, is shown in Fig. 2. The cylindrical pivot pier under the old swing bridge, which

Rolling-Lift Span 100 Ft. Long

was supported by 100 piles, was rebuilt to act as a simple truss pier, located as indicated in Fig. 1. This was accomplished by building on it a reinforced concrete cap.

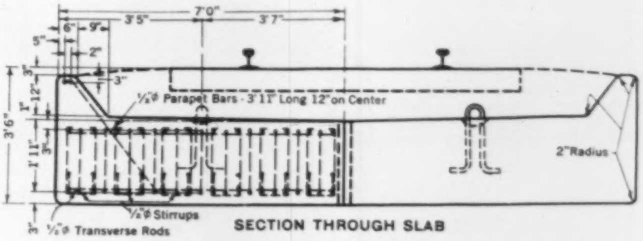
The deck-girder spans added to bridge the new overflow channel near the west end are supported on concrete piers, which rest on uncapped piles cut off below low water. No complications were encountered in the construction of these piers, and there are no features of particular interest in their design and construction. The substructure for the deck-girder spans added at the east end of the truss spans consists of 16 precast reinforced-concrete piles, octagonal in section, with faces 18 in. apart, which were driven to a predetermined depth by a heavy-duty steam pile driver using a water jet. The caps of reinforced concrete were cast in place. Collars were attached to the piles at the ground line to stiffen the pile bents, as shown in Fig. 3. The concrete piles are designed to carry a maximum load of 32 tons each.

PROVISION MADE FOR NAVIGATION

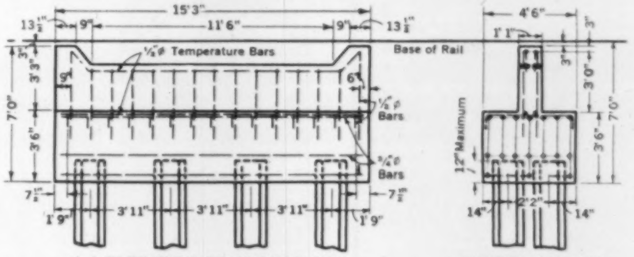
Navigation was provided for by means of a rolling-lift through-truss draw span, allowing a clear channel of 100 ft. The counterweight of the rolling-lift bridge is composed of concrete having an average weight of 310 lb. per cu. ft., obtained by using steel punchings for the coarse aggregate instead of stone. Accurate control was obtained by weighing all the materials for each batch. The lift bridge is operated by a four-cylinder 70-hp. gasoline engine and transmission unit.

The reinforced-concrete trestle approaches were built to conform to the standards in use on the Atlantic Coast Line system. Drawings of the piles and slabs are reproduced in Fig. 3. Each bent contains six octagonal piles, precast, with parallel faces 18 in. apart. Every fifth bent contains eight piles and a diaphragm for the anchorage of the deck slabs. The piles were driven to the predetermined depth with a heavy-duty track driver equipped with a steam hammer and a water jet. Calculations were based on a safe bearing capacity of 36 tons for each pile, according to Cooper's E-60 loading. The slabs, which are 18 ft. long, were precast and placed by a 75-ton locomotive crane. For that part of the trestle which is on a 6-deg. 30-min. curve, the superelevation was provided for by adjusting the length of the posts. The caps were so fashioned that the bearing contact is uniform. The bents were strengthened by collars at the surface of the ground.

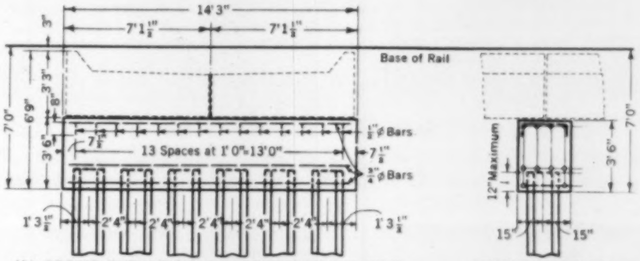
The steel superstructure was designed according to Cooper's E-60 specifications, with full impact, according to the 1920 specifications of the American Railway Engineering Association for a steel bridge. The E-40 steel span built in 1903 was not replaced, because the overstress that will be developed by the proposed locomotives will be within the safe limiting stresses recommended by the Association. All the connections were riveted, and wherever possible H-beams were used for truss members. Girder beams were employed for



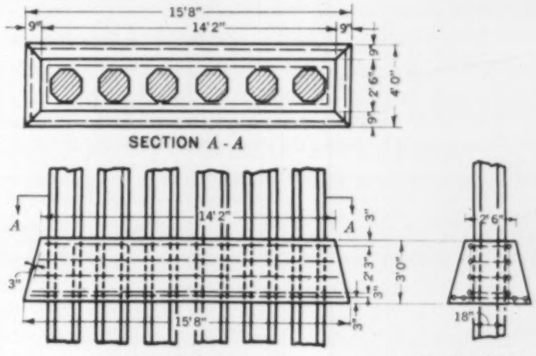
SECTION THROUGH SLAB



(a) ELEVATION OF ANCHOR PIER TYPICAL SECTION



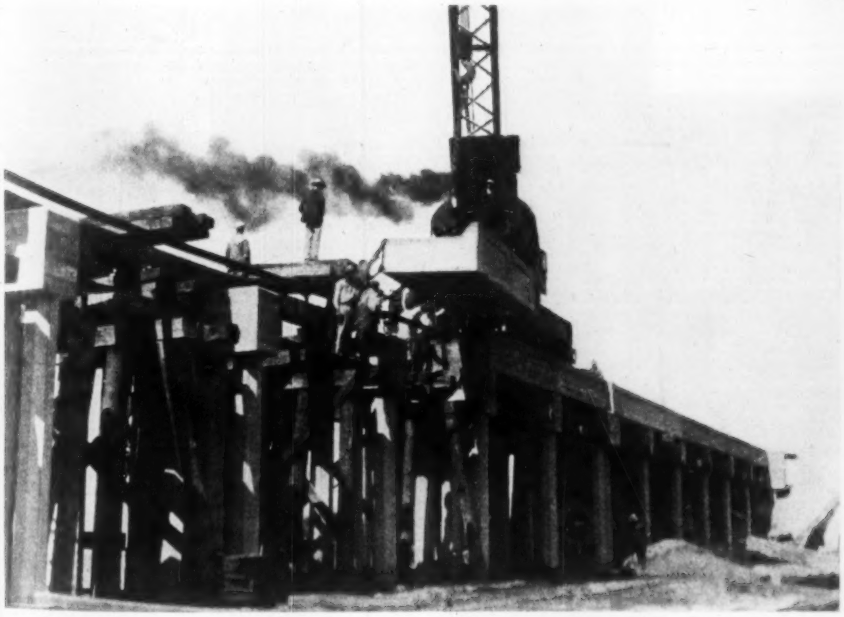
(b) FRONT ELEVATION OF INTERMEDIATE PIER TYPICAL SECTION



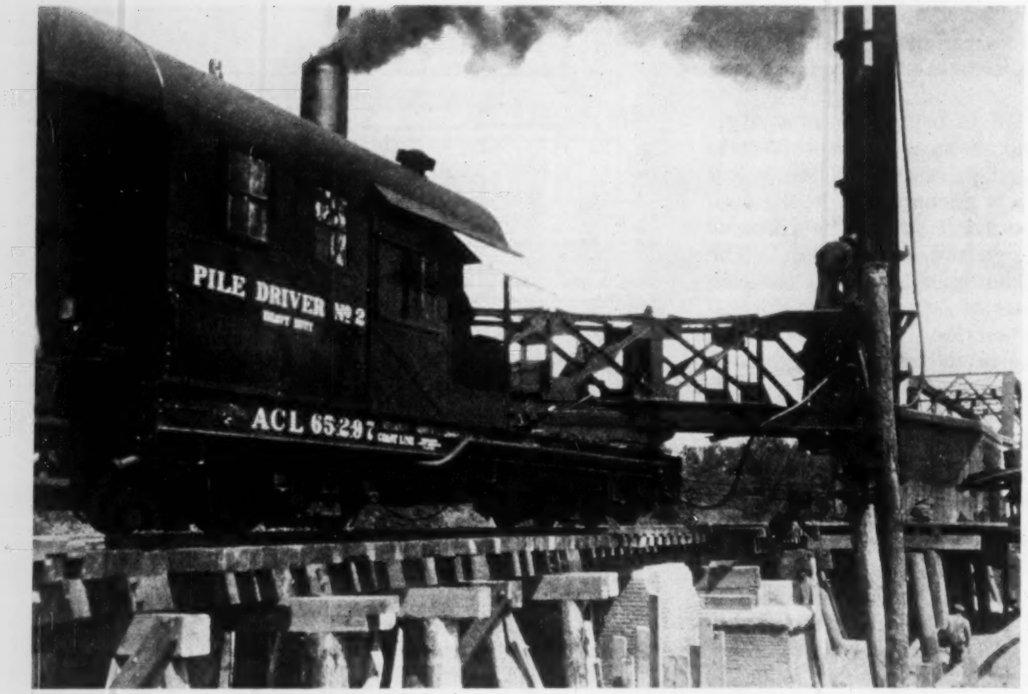
FRONT ELEVATION TYPICAL SECTION
(c) GROUND STIFFENER

Fig. 3. Details of Precast Pile Bents and Slabs for Approach Trestles

The Completed Bridge Required a Half-Mile of Precast Reinforced-Concrete Construction



Setting the Precast Slabs in the Approach Trestle



DRIVING WOODEN PILES TO STRENGTHEN FOUNDATIONS

stringers. The details of the trusses, cast-steel shoes, and truss bracing were such that all parts of the spans would be open and accessible for inspection and painting.

The bridge as reconstructed consists of the following:

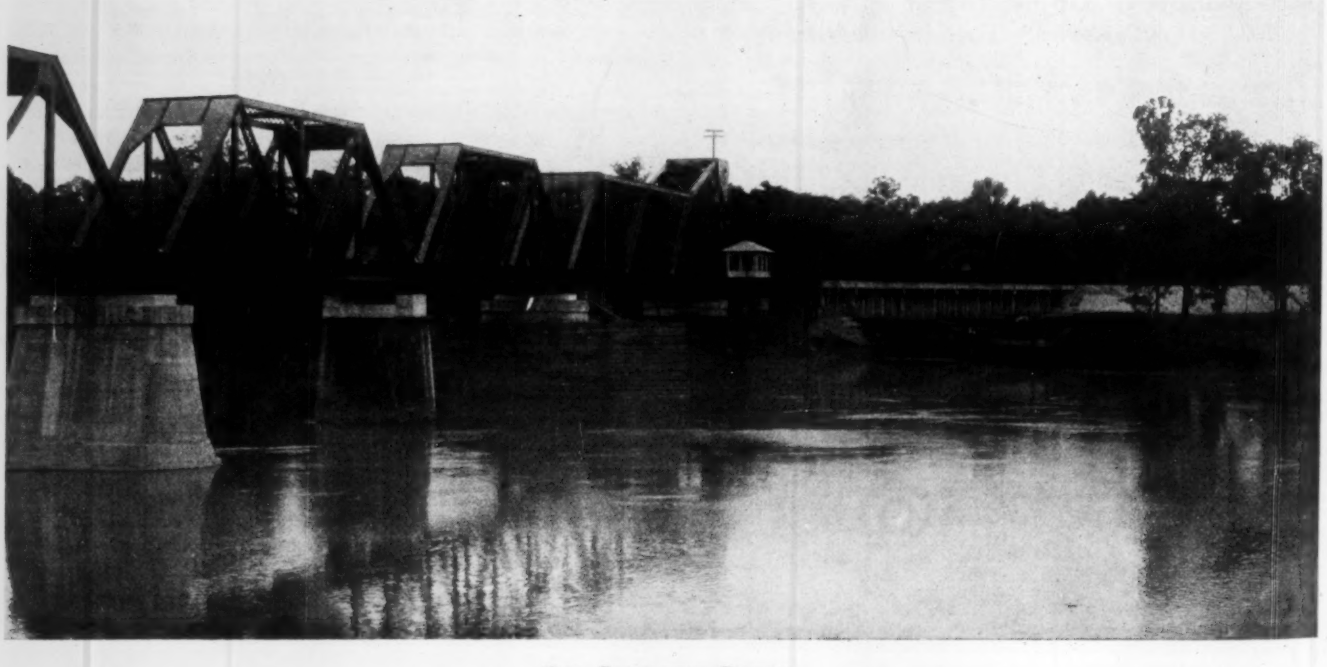
UNITS	LENGTH
Concrete slab spans	439 ft. 4 in.
Deck-girder steel spans	191 ft. 0 in.
Rolling-lift through span, including the 36-ft. tower span	191 ft. 8 in.
Through-truss steel spans	668 ft. 8 in.
Concrete-slab spans	1,664 ft. 7½ in.
Deck-girder steel spans	357 ft. 6 in.
Concrete-slab spans	219 ft. 3 in.
Total length	3.732 ft. 0½ in.

All the work was accomplished without interrupting traffic, either over the bridge or on the river. The masonry for the new piers was built to its finished height and the concrete piles were driven according to plan before the track was raised to the new elevation. Sections of the track, including the trusses, were raised from time to time on blocking as the various units of masonry and concrete piling were completed. As the sections of the track were raised, the concrete caps were built in place and the shoes for the steel spans were placed.

ACKNOWLEDGMENTS

The new structure was designed by G. G. Thomas, M. Am. Soc. C.E., Engineer of Metal Structures for the Atlantic Coast Line Railroad Company. The concrete piles and slabs were designed by R. L. Groover, Engineer of Design, and the work was supervised by G. B. Copeland, Assistant Engineer of Construction, with the aid of L. S. Jeffords, Superintendent of the Charleston and Western Carolina Railway.

The steel was fabricated by the American Bridge Company and erected by the metal-bridge facilities of the Atlantic Coast Line system, under the direction of C. T. Nelson, Supervisor of Metal Bridges. The concrete work, both precast and built in place, was carried out by the forces of the Charleston and Western Carolina Railway Company.



THE COMPLETED BRIDGE

Subdrainage of Railway Roadbeds

Increasing the Stability of the Track by Eliminating Water Under the Ballast in Fills and Cuts

By H. PHILLIP PICKERING

MEMBER, UNIVERSITY OF COLORADO STUDENT CHAPTER, BOULDER

OF the many problems confronting the maintenance engineer on railroads, drainage is one of the most important. As a part of his undergraduate work at the University of Colorado, Mr. Pickering prepared an elaborate questionnaire on subdrainage problems and sent it out to the chief engineers of all the railway companies in North and South America and England. The replies received and his own study of the problem formed the basis of a paper which he presented in February before the Student Chapter of the Society at Boulder. As noted in the September issue of CIVIL ENGINEERING, *the author was awarded the student prize of the Colorado Section for his paper. This article, a condensation of the original paper, gives the causes of water in railroad subgrades in both fills and cuts and describes methods of preventing and removing it.*

WITH the constant increase in wheel loads and traffic, attention has been more and more drawn to the problems of instability of railroad roadbeds. Higher speeds, more traffic, and greater wheel loads require more stability in the track, but these very factors have been instrumental in forming and enlarging water pockets. On most of the older lines there is a tendency toward the formation of water pockets, due primarily to neglected drainage and improperly selected ballast for increased wheel loads. Water pockets are depressions in the roadbed filled with ballast, in which water is collected and retained. The length of these pockets, measured along the center line of the track, is variable and ranges from 3 ft. up to 100 ft. or more. Numerous pockets often occur in the same fill. The profile of the bottom of them is very irregular, so that a series of small depressions may appear to be one large soft spot. In cross section water pockets will be found in general to have one of two forms, either U-shaped, as in Fig. 1 (*a*), the type that seems to be prevalent in the Eastern States, or double-V shaped, as in Fig. 1 (*b*), the more common type in the Middle West. These pockets vary in depth from 2 to 15 ft. or more depending on such factors as height of fill, wheel loads, type of soil, and age of pocket. They may even extend below the level of the natural ground, as may be seen in Fig. 1 (*b*).

The material contained in water pockets is a semi-liquid or plastic mass composed of ballast and fill material puddled with the water retained by the impervious fill. Each time the track is lifted and reballasted, the added ballast gradually sinks into the mass with the application of load. The addition of water and the continual churning under traffic cause the pockets to grow in size and depth. The amount of water retained is often astonishing. Small cavities will hold from one to three barrels of water, and larger ones when opened will produce outflows varying from 50,000 gal. to a small stream from 3 to 6 in. in diameter that may continue to flow for several days.

The section foreman first becomes aware of the presence of a water pocket when a certain section of track requires lifting and tamping every few days. The track will be brought to grade only to settle again, and "dog-legs" will appear in the alignment. This section will require attention more and more frequently, until settlement and throw may occur on the passage of a single train. The subsidence of the track will usually be 3 or 4 in. in 24 hours, the track being brought back to grade each day by the track force. Cases of extreme settlement occur frequently. On the Chicago, Burlington and Quincy Railroad, a section of fill 200 ft. long underwent a total settlement of 15 ft. A high side-hill fill on the Baltimore and Ohio Railroad settled a total of 26 ft. in a period of five months. As the track settles, bulges from 2 to 6 in. high will appear along the shoulder of the fill just outside the toe of the ballast. Similar though larger bulges may be noticed near the toe of the slope. An excessive amount of water in the fill will cause the sides to slough. The Colorado and Southern Railway Company has found that sometimes a water pocket will cause the fill to split, so that half of it will slide out. When the material in the water pocket becomes saturated, the pumping action at the joints under moving loads brings mud up into the surfacing and around the ties. In extreme cases the toe of the fill may creep, and when the pocket is so deep as to extend below the original surface, the natural ground beyond the toe may heave.

CAUSE OF FORMATION

In 1915 a committee of the American Railway Engineering Association reported that, "Our knowledge of water pockets in roadbeds is yet in its experimental stage." Since that time, however, increasing experience with these pockets and a study of their formation have resulted in the accumulation of definite knowledge regarding them. Now it is known that water is held in depressions of an impervious subgrade, that the ballast sinks into the mud thus formed, and that the additional inflow of water causes the pockets to increase steadily in size and

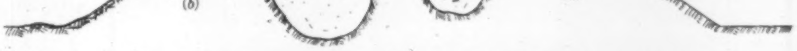


FIG. 1. TYPICAL WATER POCKETS IN RAILROAD FILLS
(*a*) U-Shape Prevalent in the Eastern States
(*b*) Double V-Shape Common in the Middle West

decrease in stability. Thus there are two primary elements in the formation of water pockets: subgrade soil and water.

The water that finds its way into depressions in the roadbed comes from both rainfall and groundwater. Precipitated moisture seeps through the porous ballast to the subgrade. On a well-crowned subgrade, water drains away to the sides, but after a roadbed has been subjected to traffic for several years, a very shallow trough develops under the rails, where the ballast has been pressed into the subgrade, and this trough has sufficient depth to direct the seepage down grade until it is trapped in a depression or in water pockets that already exist. Often a fill is down grade from a cut where water collects. Under these conditions, there will be a constant seepage from the cut along the subgrade beneath the ballast, the amount of the seepage being determined by the movement of the groundwater intercepted in the cut.

Rock or French Drains

The actual water-retaining depressions in the roadbed, or the conditions that tend to develop them, can generally be traced back to the construction of the fill. The building of a fill by dumping material from cars and shaping the surface with scrapers leaves an unconsolidated roadbed that will shrink unevenly. The laying of track for construction trains directly on a new subgrade, with no ballast, is definitely a cause of later water pockets. Although the apparent depressions and tie-cuts are filled before ballasting, the material used will be less compact than the rest of the fill, and the hollow will be permanent as far as its capacity to retain water is concerned. Similarly, when fully ballasted track is laid on a soft roadbed that has not had time to become sufficiently compacted to resist the pounding of traffic, ballast-filled pockets will result.

In climates where freezing temperatures occur, frost action is another cause of water pockets. Since soil thaws from the top down, there will be alternate thawing and freezing of the upper layers while the soil below remains frozen. During thawing, the semi-liquid mass directly beneath the ballast section will be supported by the frozen soil below, through which the water will be unable to pass. The passage of traffic will churn this mud, forcing the ballast down into it and pushing the muck to the sides, thus starting a water pocket.

For some time it was believed that the tamping of ballast under the ties when bringing the track to surface was a cause of depressions in the subgrade, but it has since been concluded that tamping and surfacing methods have no effect. However, the depth of the ballast does have a decided effect on the character of the subgrade surface. A ballast section that is too light to uniformly distribute the increasingly heavy wheel loads over the surface of the subgrade, will cause the track to roll and will result in the formation of water-retaining hollows in the valleys.

The methods of treating water pockets in through fills are necessarily dependent on the character of each individual pocket and the conditions under which it occurs. Type of soil used in the subgrade, character of the natural ground, geology of the location, source of the water, type of ballast, density of traffic, wheel loads, size and shape of the pocket—all these factors must be borne in mind. What may be a remedy for one water pocket may be of no use for another, even though the two may be in rather close proximity. Treatment often only relieves the condition and does not effect an absolute cure.

Before making an estimate for a drainage job, a line map and profile of the track through the affected area should be drawn up. This map must show the shape and extent of the water pockets, as well as profiles of the bottom of them. The accurate location of the bottom of the water pockets is absolutely imperative. The outline and profile of the pocket may be determined by taking frequent soundings with the earth auger, or often, by digging near the ends of the ties with a shovel. The water standing near the surface gives a quick indication of the extent as well as something of the character of the pocket. The topography of the natural ground should be shown in order that the outlet may be accurately located. The possible source of the water and other helpful topography should be noted. All this informa-

Perforated Corrugated Pipe Laid in the Bottom of a Rock-Filled Trench tion is useful in determining the type of drain to be used and in preparing the estimate.

FRENCH DRAINS EFFECTIVE

The most generally used method of treating water pockets is to install French, or rock drains, as shown in Fig. 2 and in two of the photographs. A trench from 4 to 8 ft. wide is dug at right angles to the center line of the track, from the toe of the fill through the lowest point in each pocket. When the pocket is largest under the near rail, the trench is extended to a point beneath the far rail, and when the pocket extends deeply under both rails, the trench is extended through the entire fill. When much water is encountered in the pocket, it is necessary to allow time for drainage before the digging of the trench is continued. The bottom of the trench must be well below the lowest point of penetration of the pocket; otherwise the drain is useless and the entire cost is wasted. The trench is carried at a grade of from 4 to 6 per cent.

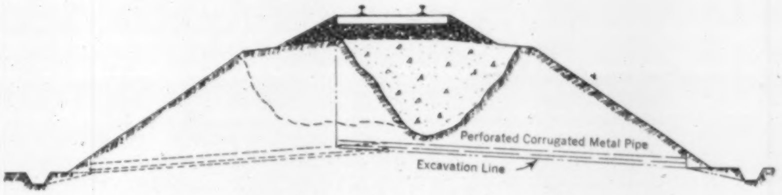


Fig. 2. Method of Draining Water Pockets
Using Perforated and Corrugated Metal Pipe

At the bottom of the trench a perforated metal pipe from 6 to 10 in. in diameter is laid, with the perforations down, because water enters the submerged pipe from the bottom. The pipe sections should be joined with metal collars so that there will be no displacement of the joints and failure of the drain in the event that future settlement occurs. The end of the pipe under the track must be plugged to prevent clogging. After the pipe is laid, the trench is back-filled with coarse broken stone or "one-man" riprap. The stone is laid with the largest sizes near the bottom and is graded upwards.

Some indication of the cost of individual drains may be obtained from the records of the Chicago, Burlington, and Quincy Railroad, on which this cost ranged from $10 to $50. Of course these drains were not very extensive in character. In one instance the total cost of remedying an extremely troublesome pocket was $2,500, including shoring the excavation and cribbing the tracks.

The effective age of rock, or French, drains is not well established. Many have given service for between 10 and 20 years, and some of the first ones, installed about 30 years ago, are still accomplishing their purpose.

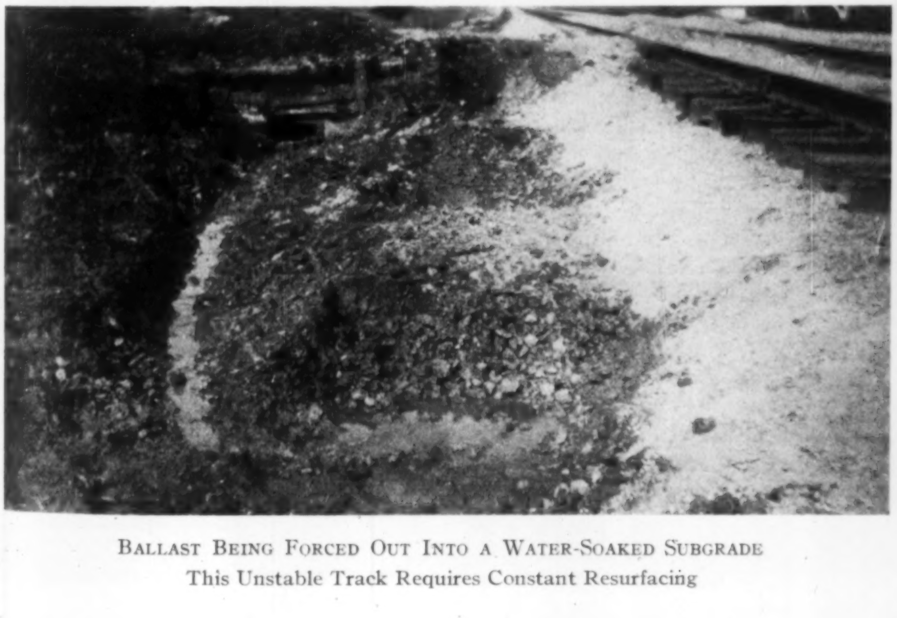
It has been found that the profile of the bottom of water pockets has been accurately determined by driving pipes horizontally into the embankment. Old locomotive boiler flues have been used thus by several railroads, including the Chicago, Burlington, and Quincy. The New York Central Lines have tried jacking metal pipe into the fill. A machine that will jack corrugated and perforated metal pipe into fills to tap water pockets has been developed for drain pipe between 8 and 10 in. in diameter. This unit consists of an earth auger attached to a spiral conveyor working inside a casing, which in turn is inside the corrugated pipe. The conveyor removes the excavated material as the pipe is jacked forward into the fill.

An interesting example of the use of perforated metal pipe to stabilize a soft roadbed is the method adopted to treat a serious situation in an approach fill of the bridge over the Missouri River near Sibley, Mo., on the main line of the Santa Fe Railway (Fig. 3). The fill, about 52 ft. high, had been built up of sandy clay. This material was dumped into the fill directly from dump cars, and the surface was then graded off. The ballast used was gravel. Water pockets from 6 to 12 ft. deep eventually developed and caused the fill to slough and slide so badly that traffic was interrupted. About 20 slides developed in a distance of 5,000 ft. To remedy this condition a trench $2^1/_2$ ft. wide was dug between the tracks and extended below the bottom of the water pockets. Perforated metal pipe 6 in. in diameter was laid on 4 to 6 in. of crushed stone. Perforated laterals 13 ft. long were placed every 50 ft. on an 18 per cent grade. At intervals of 500 ft., discharge pipes led to natural ground. These were provided with concrete headwalls. The back fill was of crushed stone. The main drain was about 3,500 ft. long.

Occasionally an impervious fill will be built on undisturbed ground that is sandy and porous. Water pockets in such a fill have been drained by the New York Central Railroad by driving and withdrawing round piles vertically at the ends of the ties, to puncture the impervious material and lead the water downward into the porous soil. The Chicago and Northwestern Railroad has used vertical drains driven to below the groundwater level to carry off the troublesome surface water.

When a water pocket develops in a fill, the only real remedy is to remove the water. Other methods have been tried with more or less success but until the pocket is drained and kept so, there will be continual trouble with the track. However, varying degrees of relief have been effected by some of the following methods.

Of all the temporary relief mea-



Ballast Being Forced Out Into a Water-Soaked Subgrade
This Unstable Track Requires Constant Resurfacing

sures, the use of piling to support the track is probably the most extensive. At first glance it would appear that piling driven at the ends of the ties would hold the fill in place, but the nature of the material, made plastic by the presence of water, allows the fill to flow around and past the piling. It is not at all uncommon for the moving earth to carry the piling with it. On one bad section of the Chicago, Burlington, and Quincy Lines, piles were moved as much as 30 ft. from the point where they were driven. Piling will support a badly settled section and retard heaving of the sides, but only for a short period. It must therefore be considered as only a temporary measure. Extreme cases of settlement on the New York Central Lines have been successfully cured by the construction of submerged pile trestles.

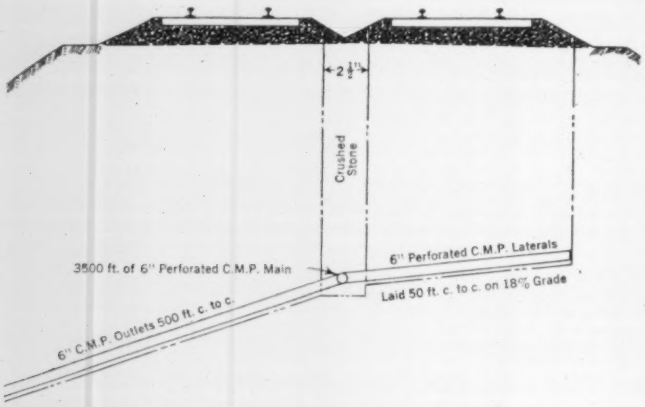

2½"
Crushed Stone
6" Perforated C.M.P. Laterals
3500 ft. of 6" Perforated C.M.P. Main
Laid 50 ft. c. to c. on 18% Grade
6" C.M.P. Outlets 500 ft. c. to c.

Fig. 3. Santa Fe Railway Stabilizes a Long Fill at Sibley, Mo.

3,500 Ft. of Drains, Installed in the Approach Fill to the Bridge Over the Missouri River

Wooden mats, usually made of old ties, have been placed beneath the track. These mats tend to float the track and eliminate pumping at the joints. Concrete slabs have also been employed in this manner. In a few instances where it has been determined that the flow of water follows the fill down grade, vertical cut-off walls of sheet piling or concrete have been installed. An adequate system of drain pipes must necessarily be placed on the upstream side of such walls so that water will not be impounded.

RESULT OF CAREFULLY BUILT FILLS

To illustrate the results that might be achieved by careful construction methods coupled with an understanding of the nature of water-retaining depressions, the Bessemer and Lake Erie Railroad carried out an experiment in 1910. Several miles of a single-track line were to be made double track, a procedure that required the widening of some long fills. Clay and shale were the materials used. One section was built by dumping the material from cars, grading, and placing the ballast and track. A second section was built in layers. The material was dumped from cars and compacted by a ten-ton roller, which was driven back and forth over the section. Any soft spots that appeared were plowed and re-rolled. The top of the fill was well crowned and rolled until a hard crust was formed. In due time the roadbed was ballasted, the track was laid, and the line was opened to traffic. The test sections carried a dense traffic and very heavy wheel loads. Six years after construction a committee of the American Railway Engineering Association investigated the two sections.

The first section had developed water pockets and required frequent attention to keep the track in line and up to grade, whereas the second section drained well, was free from water pockets, and required little attention from the track force.

WET CUTS A PROBLEM

In cuts as well as in fills, an unstable roadbed will result from an accumulation of water that keeps the subgrade moist and plastic. A water-retaining depression in a cut differs from a water pocket in a fill, in that it usually extends throughout the entire length of the cut. Geological formations or sudden changes in the soil strata will alter this, breaking the depression into several pockets. The material beneath the ballast section, as in a water pocket, is a semi-liquid mass composed of ballast, soil, and water. A greater percentage of water will be found in cuts than in fills, if taken on a yearly average. The subsidence of ballast in cuts will extend from 2 to 8 ft. below the subgrade.

In rock excavation the shattered rock and uneven breakage will facilitate the collection of water. Where the cut intersects water-bearing strata, as in Fig. 4 (*a*), springs feed water into the cut during their seasons of flow. Aside from springs, perhaps the most general source of water in a cut is groundwater, where the cut has been carried below the original groundwater level or this level is close to the bottom of the ties, as in Fig. 4 (*b*). In such cases groundwater furnishes a supply which continually saturates the lower stratum of the subgrade. Most of the other factors causing the retention of water in cuts and the resulting soft condition of the roadbed are identical with those discussed under fills.

In cut as well as in fill, too much stress cannot be laid on an accurate location of the lowest point of penetration of the water-retaining depression. A drain of any type is an absolute waste of money if it is not placed at the depth necessary to tap the lowest cavity. A study should be made of the geology of the site to determine, if possible, the source of the water that has made the section unstable.

The one method of curing wet cuts that is used by railroads throughout the United States and Canada is the installation of a subdrain system beneath the water-filled depressions, consisting of mains laid parallel to the center line of the track and laterals placed at right angles to them at frequent intervals. The two kinds of pipe used are vitrified clay sewer pipe and perforated corrugated metal pipe. Clay pipe is the most extensively used, but corrugated metal pipe has the advantage that it is not as easily displaced by earth movements. The effective life of clay pipe is influenced by subsidence of subgrade

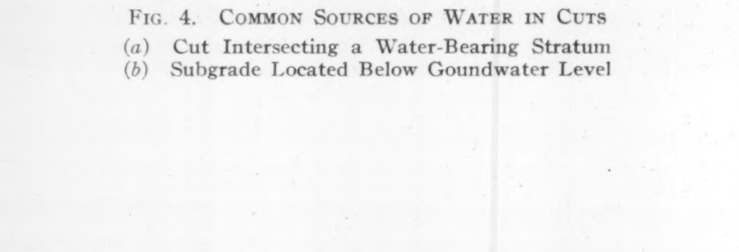
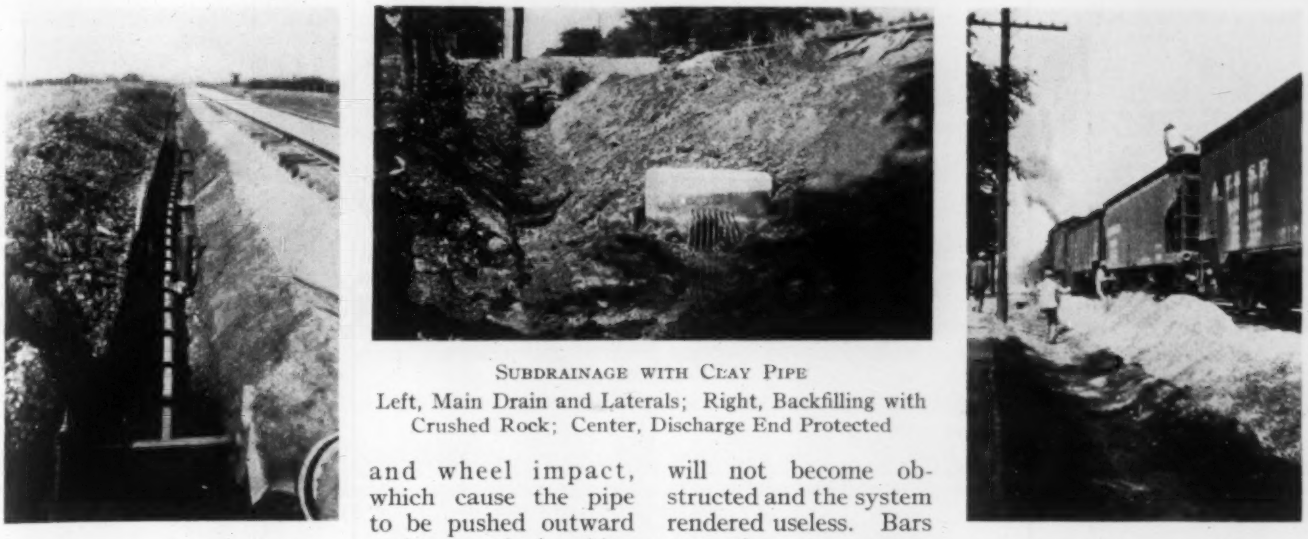


Fig. 4. Common Sources of Water in Cuts

(*a*) Cut Intersecting a Water-Bearing Stratum
(*b*) Subgrade Located Below Goundwater Level



Subdrainage with Clay Pipe
Left, Main Drain and Laterals; Right, Backfilling with Crushed Rock; Center, Discharge End Protected

and wheel impact, which cause the pipe to be pushed outward and upward, breaking the joints. The metal pipe is easily placed and there is no loss from breakage.

INSTALLATION OF METAL AND CLAY PIPE

The installation for both kinds of pipe is similar, with the exception that in some instances the metal pipe is jacked, as previously explained. The placing of clay pipe will therefore be described here. The trench for the main is dug parallel to, and at a distance of from 8 to 12 ft. from, the center line of the track, depending on the section of the roadbed. For a single-track section, the main is placed on the upstream side of the roadbed to act as an intercepting drain to carry away the groundwater as it seeps toward the roadbed. Where there is a double track, a main is usually placed on each side of the roadbed. In multiple-track sections, mains are placed between the tracks. The depth of the mains must be the depth of the lowest depression plus the drop of the laterals.

The tile is laid directly on the bottom of the trenches except where the bottom is very soft and wet, in which case the tile must be laid on a plank floor to maintain an even grade and unbroken joints. In rock excavation, a bed of tamped clay must be prepared.

Mains are made of 8 to 12-in. pipe, laid on a minimum grade of 0.3 per cent, a 0.6 per cent to 0.8 per cent grade being used where possible. At intervals of from 15 to 25 ft. laterals extending to beneath the farther rail are laid at right angles to the main. The laterals are of 6 to 10-in. pipe, laid on a grade determined by the depth of the depression and the location of the main. Their open ends are fitted with perforated vitrified clay disks. A group of photographs illustrates the general method employed.

MATERIAL FOR BACK FILL

Back-fill material may consist of crushed stone, gravel, or furnace slag. Cinders mixed with crushed stone or gravel, but not with slag, will give the drain a longer effective life than stone or gravel alone. Cinders should not be used alone as they will not withstand earth pressures and will allow the trench to distort, nor should they be used where metal pipe is installed. The back filling should be placed by hand up to the top of the pipe so that the pipe will be held firmly in place.

The outlet of the main must discharge into natural drainage, and must be at such an elevation that backwater from flood flows will not reach it. Some type of headwall should be placed at the outlet so that the main will not become obstructed and the system rendered useless. Bars or grating are necessary to keep rodents and small animals out of the pipe.

The cost of installing a tile subdrain system depends on the conditions under which it is constructed. This cost will vary from $0.40 per lin. ft. of drain to as much as $3 per lin. ft., the average being $2. The higher cost is made necessary by excavation in rock cuts where springs follow the porous strata.

The benefits to be derived from an improved roadbed that has been stabilized by the construction of an adequate subdrain system cannot be easily estimated in terms of dollars and cents. But every superintendent knows only too well the cost of supplying partial carloads of ballast to isolated localities between switching points. The switcher and crew must be provided to take the material to the affected section, the material must be unloaded, and the necessary stand-bys must be made to avoid interference with routine traffic. Load after load of ballast will be placed under the track, only to disappear from sight in a short time—money thrown into quicksand, as it were. The saving in ballast and the labor in surfacing will often amount to 75 per cent of the maintenance on the affected area.

SUBDRAINAGE EFFECTS SAVING IN TIES

The replacement of ties carrying the rails over water-filled depressions of the roadbed causes a cumulative maintenance cost. Water-soaked ties, which rot quickly and have their spikes pulled out by rail movement, must be taken out of the track. Ties that have been center-bound by the heaving of the subgrade do not provide the proper support for heavy loads. Many ties are rendered unsafe as a result of the mechanical wear caused by frequent and excessive tamping in the attempt to maintain the proper grade and alignment. Also, continual abrasion of the ties results from movement up and down against rough ballast under the passage of traffic. With the elimination of these causes of deterioration of ties, another saving in material and labor is realized. Competition for passenger traffic is keen among railroads, and the railroad that can offer the comfort and the safety which come with a smooth, unresisting roadbed, has a decided advantage.

To the chief engineers of the railroads of the United States and Canada is extended my appreciation for their graciousness in so adequately answering the questionnaires that were sent to them. C. A. Gates, Engineer for the Atchison, Topeka and Santa Fe Railway Company, supplied material that made much of this article possible.

Limiting the Size of Buildings

Their Height and Density Considered in Relation to the Horizontal Movement of Traffic

By Thomas Adams

Consultant to the Regional Plan of New York and Its Environs

GREAT metropolitan cities, such as New York, Paris, London, and Berlin, are finding difficulty in keeping step with the increasing concentration of city population and in transporting their population to and from the many places of business. In New York the convenience and economy of the crowded skyscraper are being weighed against the inconvenience and high cost of getting to and from it. As the result of many years of study, Mr. Adams, assisted by the competent staff of the Regional Plan of New York, concludes that skyscrapers are both necessary and desirable but that they must be so dimensioned in comparison with the size of the plot on which they stand, that each will have adequate light, view, and air. Also, their aggregate cubage must not be greater than the traffic capacities of the avenues by which they are bounded. In this article, prepared from a paper presented on March 24, 1932, before the Metropolitan Section of the Society, Mr. Adams gives the maximum dimensions recommended by the Regional Plan for buildings in the various sections of the Metropolitan area and considers future possibilities in the way of arcaded and elevated sidewalks, two-level streets, and separated skyscraper towers.

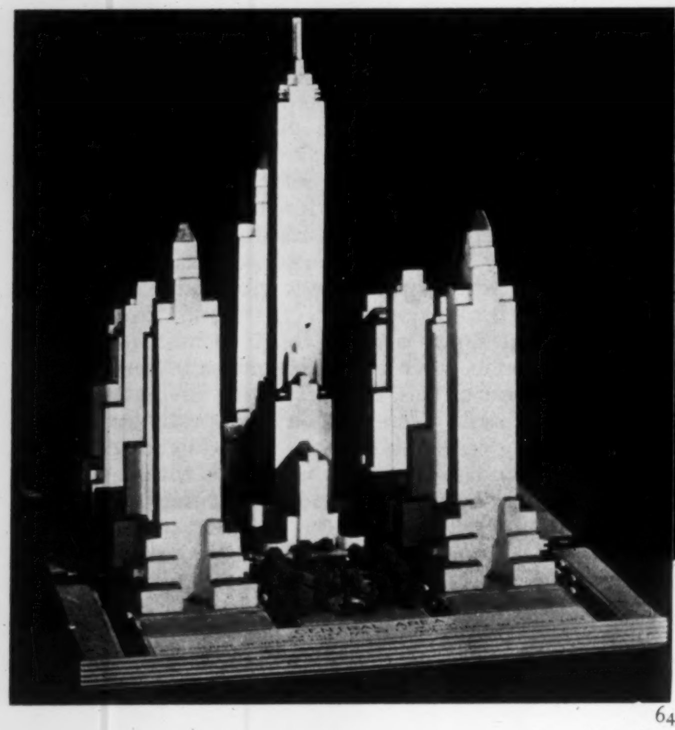
IN determining the proper height or degree of density of buildings, the basic economic consideration is that the services of locomotion—of traffic in all its forms—must be provided with efficiency and at a reasonable cost.

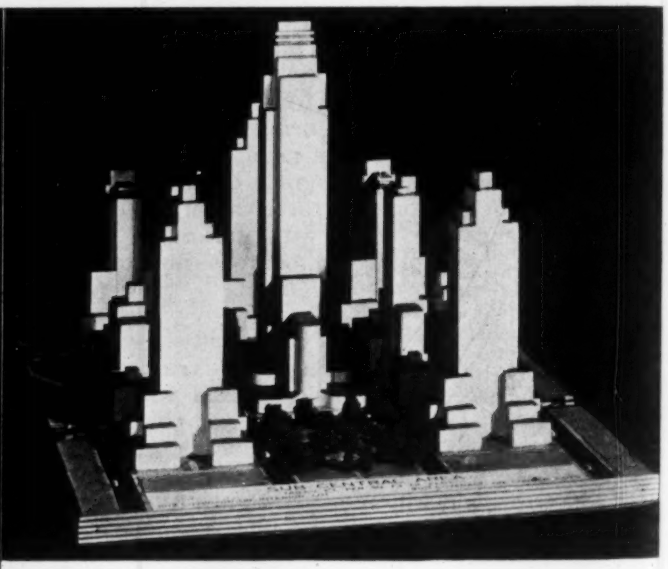
Efficiency in locomotion should be measured in terms of freedom as well as rapidity of movement. This combined freedom and rapidity is, on all counts, the major factor in building up and maintaining the prosperity of a community. Whatever impairs it, impairs the future of the community. The determination of what is a reasonable cost for traffic and transportation facilities can only be reached after studying a series of intricately related factors, but such a study has never been made. As regards railroads, transit lines, and public vehicles, it is a fair assumption that a reasonable cost for the service is that which can be met out of the direct returns in passenger fares or freight rates. As regards streets, although their uses are numerous and complicated, it is fair to consider a reasonable cost for their construction to be that which can be met by taxation on abutting or benefited property, plus taxation on vehicles using the streets. In both cases, the tax should be such as can be met by the income obtained from the use of the property or vehicle without injury to that use.

A well-planned city is one that includes, among other things, an efficient and economical system of locomotion, founded on the foregoing considerations. Many cities in the past and some small cities today are well planned from this point of view. Probably every great city, with over a million population, is ill planned and therefore constitutes an example of failure to meet the test of efficiency and reasonable cost in connection with traffic and transportation facilities.

LIGHT, AIR, AND SPACE FOR RECREATION

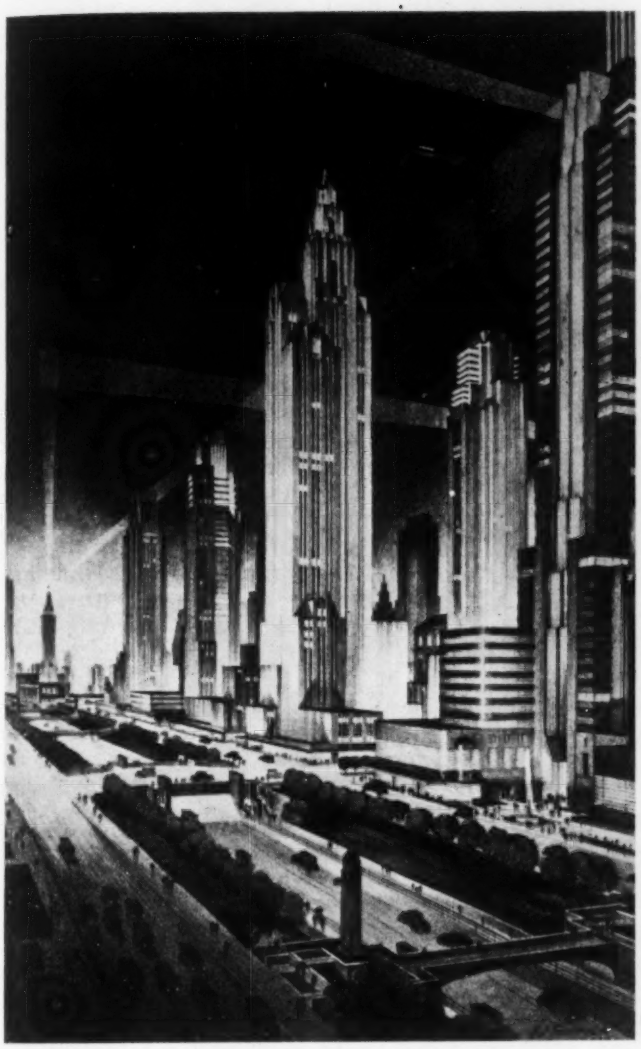
Light, direct ventilation, and space adjoining buildings for outdoor recreation are essential for the efficient and economic development of a community. But important





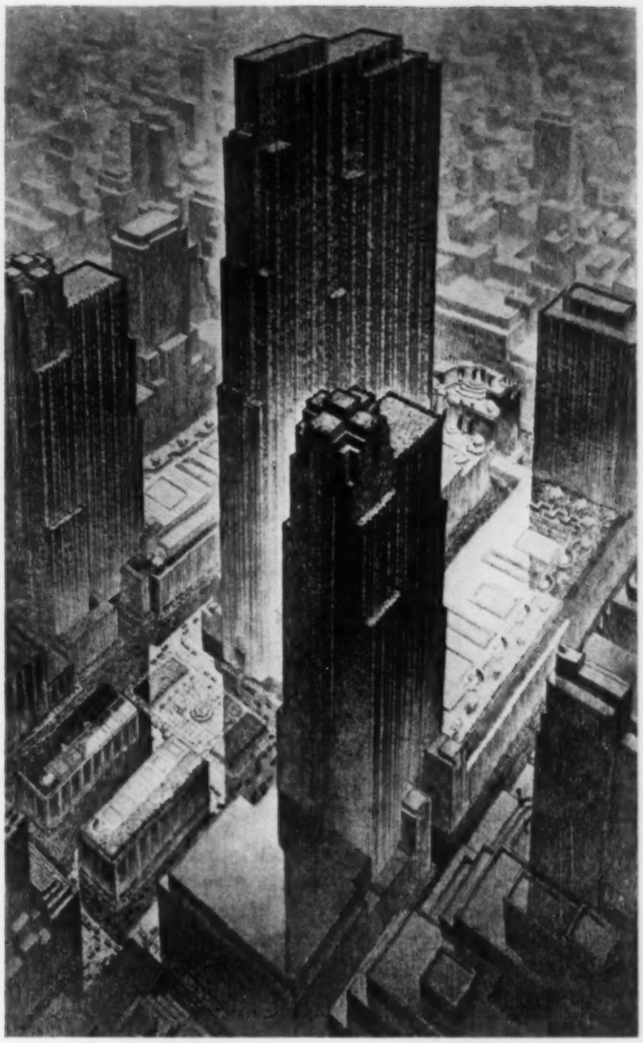
Models Showing How Intensively Developed Areas of the Metropolitan District Might Appear Under the Zoning Standards of the Regional Plan

Left: Central Area Above: Sub-Central Area



Regional Plan Proposal for the Development of the Chrystie-Forsythe Area

Open Spaces Around Buildings Assure Adequate Light and Air



Architect's Sketch of Rockefeller Center, Now Under Construction in New York

Example of Design to Provide Space for Light, Air, and View

as these factors are, I pass them over as secondary because, if adequate open space is allowed in lots and parks to keep building densities within proper limits in relation to traffic requirements, and if sufficient room is also provided in streets for traffic on one level, there will be a reasonable amount of space for all other purposes. In any event, these other purposes should be dealt with by zoning regulations. No reason for restricting the heights of buildings on other grounds than those relating to movement—a matter ignored in zoning—will be presented here.

PROPORTION OF OCCUPIED TO OPEN LAND

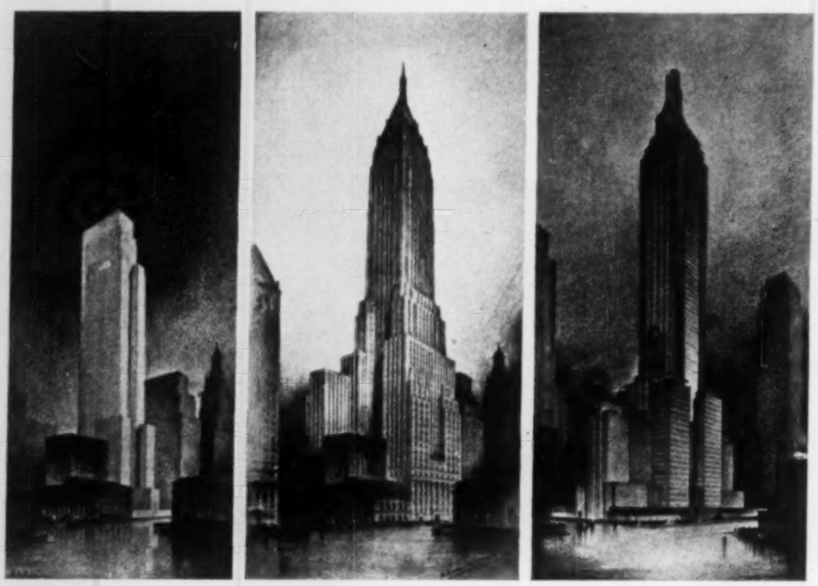
In the Regional Plan of New York it is suggested that there is much more land in all parts of the region than will ever be wanted for building purposes. There can be no question but that industry, residential interests, and business can have all the space they want. The apportionment of this space to meet social demands is again the responsibility of those who provide transportation, rapid transit, and highway facilities. Excessive concentration, that is congestion, is in the main the result of badly arranged facilities for horizontal movement.

The estimate of the Regional Plan is that, in areas where the erection of high buildings is possible, no more than 50 per cent of the land should be built on even in central areas; that is, 50 per cent of the land should be private and 50 per cent public. Of the private land, only 80 per cent should be covered by buildings, or 40 per cent of the total area. Public land in such areas should be allotted in the proportion of 40 per cent to streets and main highways and 10 per cent to parks and parking areas.

If these percentages are not available on the level they must be provided by decking the streets. In part the arguments for these proportions are based on conditions in the most intensively developed area of New York. In the section south of Fulton Street, 48 per cent of the land is occupied by buildings; and there are numerous courts on this private land. This area contains many high buildings and is as intensively occupied as it should be. Between Fulton Street and 59th Street the land occupancy increases to 61 per cent, which is excessive.

RECOMMENDED HEIGHTS OF BUILDINGS

In the Regional Plan the skyscraper is accepted as both necessary and desirable, but the crowding together of skyscrapers is deprecated as a cause of congestion. In itself, height is only to be deplored when it is a factor in creating excessive bulk or cubage in relation to the traffic capacity of the area. As already stated, the test of the proper building density is whether or not it provides adequately and economically for locomotion. Strictly from the point of view of the architect and real estate operator, it is immaterial how high the buildings are; what is important is that this height should be economically sound from the point of view of traffic movement. Present buildings are already passing the practical limit of height and density. Recent amendments to the New York City zoning ordinance have increased the permissible height at the street line in certain sections of Manhattan form $1^{1}/_{2}$ to 2 times the street width. That building can continue to the extent foreshadowed by the existing law and such modifications, without any control of bulk, is an idle dream.



BANK OF MANHATTAN BUILDING, WALL STREET AS ACTUALLY CONSTRUCTED (CENTER)

As It Would Appear if Built According to the Limitations of the Regional Plan on the Same Site (Left), and of the Same Cubage on an Adequate Site (Right)

As has been shown by Daniel L. Turner, M. Am. Soc. C.E., it is probably impracticable to increase transit facilities sufficiently to allow for an extension of the densities of buildings now found in the main business centers. If all the blocks in midtown Manhattan were improved with buildings built up only to the height at the street line permitted by the zoning ordinance, that is, without setbacks or towers, the resulting traffic would probably be almost twice the estimated capacity of the existing avenues. But we are told that it is more practicable to double or treble the street capacity than to impose more limits on height. Time will tell, but I believe that the cost of obtaining the required capacity will be found to be prohibitive.

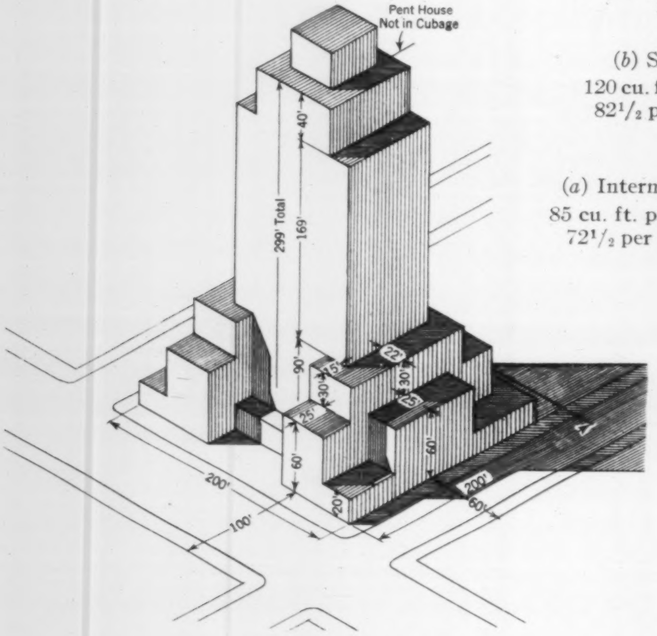
In the area south of Central Park (59th Street) in Manhattan, the average building height per unit area of plot is 6.1 stories, including the average of 10.7 stories in the area south of Fulton Street. Between Houston and 14th streets it is only 4.9 stories. In 1929 there were 1,423 buildings of over ten stories south of 59th Street. A summary of the present building situation in Manhattan south of 59th Street is given in Table I.

The Regional Plan classifies business areas as central, sub-central, intermediate or close suburban, and open suburban. The standards for maximum building density may be briefly stated as follows:

> In central areas, which are those now likely to be covered with buildings of a height at the street line greater than $1^{1}/_{2}$ times the width of the street, without any limit on bulk, an effort should be made to limit the height at the street line to 60 ft., then to require a setback of 25 ft., above which a total height of 200 ft. would be allowed. On 20 per cent of the lot, a tower of unlimited height should be permitted, with restrictions as to setbacks; but the cubical contents should be limited to the equivalent of 12 floors solid on the total area of the lot, and this would indirectly restrict

FIG. 1. RECOMMENDED HEIGHT AND SETBACK

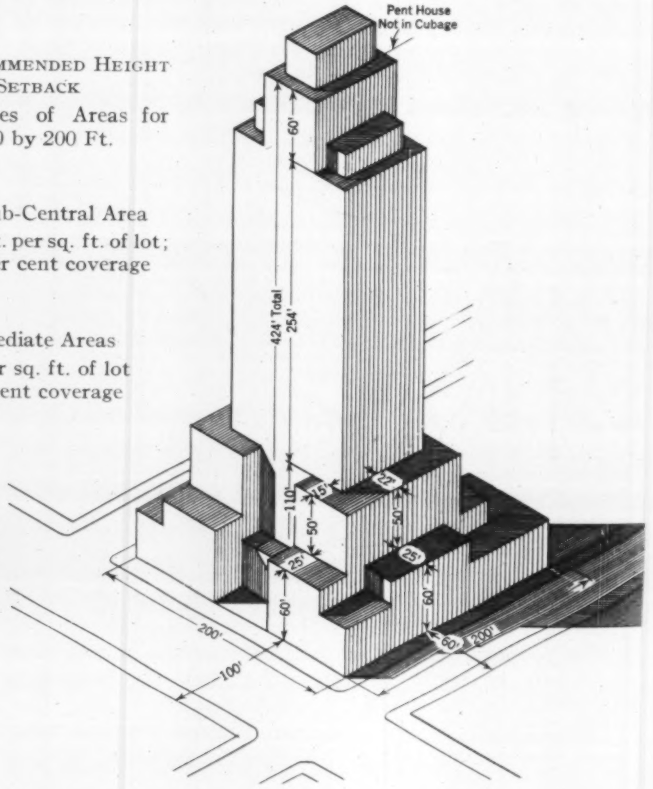
Different Types of Areas for Plots 200 by 200 Ft.



(a) Intermediate Areas

85 cu. ft. per sq. ft. of lot

$72^{1}/_{2}$ per cent coverage



(b) Sub-Central Area

120 cu. ft. per sq. ft. of lot;

$82^{1}/_{2}$ per cent coverage

the height of the tower. Coverage would be limited to between 80 and 90 per cent.

It is absurd that this regulation should be considered impracticable. Sooner or later attention must be given to the requirements for horizontal movement and to a limitation in the amount of dark space permitted in buildings.

Table I. Building Heights and Ground Occupancy
South of 59th Street, New York

Section	Percentage of Area		Buildings of Over 10 Stories	Average Height in Stories per Unit Area of Plot
	In Streets and Parks	In Building Sites	(in 1929)	
South of Fulton street	52	48	142	10.7
Fulton to Houston streets	41	59	85	5.1
Houston to 14th streets	39	61	82	4.9
14th to 34th streets	39	61	456	6.0
34th to 59th streets	38	62	658	7.0
Total or average south of 59th Street	39½	60½	1,423	6.1

In sub-central areas, roughly those in which the height of buildings at the street line may now be greater than 1 times but not in excess of 1½ times the width of the street, the limit of 60 ft. up to the first setback is again proposed, beyond which buildings may rise to 140 ft. Towers should be permitted on 20 per cent of the lot, and the cubical contents should not exceed the equivalent of 10 floors solid. Coverage should be limited to between 75 and 90 per cent of the lot.

In intermediate areas, the height limit proposed is also 60 ft. up to the first setback, above which buildings may rise to a total height of 120 ft., the allowable percentage of the lot for towers to be 20 per cent, and that for coverage, between 70 and 80 per cent.

In open suburban areas the height limit of 60 ft. at the street line is again recommended. A setback of 25 ft. is then proposed, above which any additional height is permitted, including a tower, that will maintain a 45-deg. angle of light to the front and rear of the building. The cubage suggested is that of 6 or 7 floors, equal in size to the section of the lot that can be built upon.

All these proposals, illustrated in the photographs of prepared models, are supplemented by other proposals regarding front and rear yard space and variations applicable to residential buildings and special circumstances.

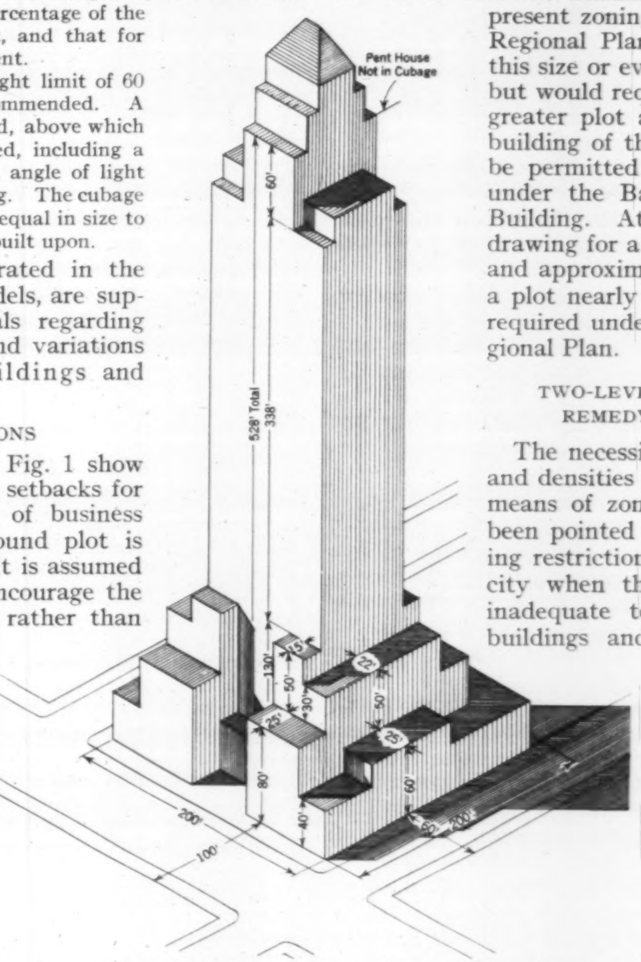


Fig. 1 (c). Central Area, 144 cu. ft. per sq. ft. of lot; 82½ Per Cent Coverage

PRACTICAL LIMITATIONS

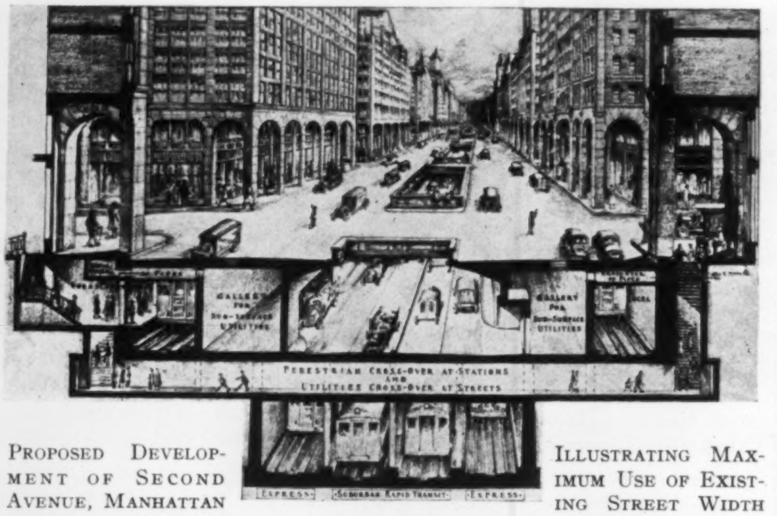
The isometric drawings in Fig. 1 show the recommended height and setbacks for buildings in different types of business areas. In each case the ground plot is taken to be 200 ft. square. It is assumed that present tendencies will encourage the placing of bulk in the tower rather than in the lower part of the building. The greatest attainable height for a single block in central areas, as shown in Fig. 1 (*a*), is 528 ft. Towers 800 ft. or more high should be erected only as part of a plan to develop two or three blocks together, so that the tower can be placed on one block and lower buildings on the other. Rockefeller Center, now under construction between Fifth and Sixth avenues, from 48th to 51st streets, is a good example of this arrangement. An architect's conception of the completed group of towers, here reproduced, shows how the isolated grandeur of the towers is enhanced by the open spaces left for light and ventilation.

The Regional Plan has endeavored to arrive at a reasonable compromise between the bulk permitted by the present zoning law and that regarded as ideal from the point of view of light and traffic movement. From every point of view except that of what the public will stand for, the proposals of the Regional Plan are vastly more practicable than the existing zoning law. Of course, what is now practicable is only what the public, the Board of Estimate, and the courts will accept at this time. Considered from the political and legal standpoint the proposals of the Regional Plan are admittedly an ideal dream. But public opinion, boards of estimate, and courts may modify their views to suit changed conditions. It seems inevitable that the growth of business and resident population, and consequent transit congestion with the increase in building density, and the limits to the burden of municipal taxation, will show in course of time that the dreamers are those who permit building to go on unchecked without attempting to estimate the cost of providing the essential transportation facilities.

In the center of one of the accompanying illustrations is shown the Bank of Manhattan Company Building on Wall Street, one of the finest examples of present-day skyscraper architecture. The part of the building below the tower has the maximum bulk permitted by the present zoning law. The proposals of the Regional Plan would permit buildings of this size or even larger in central sections, but would require that they have a much greater plot area. At the left is shown a building of the maximum bulk that would be permitted on a plot the size of that under the Bank of Manhattan Company Building. At the left is reproduced a drawing for a building of the same height and approximately the same bulk, but on a plot nearly twice as large, as would be required under the proposals of the Regional Plan.

TWO-LEVEL STREETS PROPOSED AS REMEDY IN CERTAIN SECTIONS

The necessity of adjusting building uses and densities to established street areas by means of zoning restrictions has already been pointed out. But whatever the zoning restrictions, the time comes in every city when the original street system is inadequate to serve both the abutting buildings and the through traffic. The proposals of the Regional Plan for streets or sidewalks on two levels are put forward to remedy conditions for which no better solution is possible. Since the Regional Plan frankly admits that the situation existing in certain skyscraper districts cannot be remedied by zoning, it must also accept the two-level street system. But this does not affect the


Gallery for Sub-surface Utilities
Gallery for Sub-surface Utilities
Pedestrian Cross-Over at Stations and Utilities Cross-Over at Streets
Express
Suburban Rapid Transit
Express

PROPOSED DEVELOPMENT OF SECOND AVENUE, MANHATTAN ILLUSTRATING MAXIMUM USE OF EXISTING STREET WIDTH

Depressed Express Highway and Arcaded Sidewalk Within the Building Line

soundness of the general rule that one-level streets are preferable to two-level streets.

Examples of two-level streets are illustrated in the Regional Plan proposal for the strip recently cleared of slums between Chrystie and Forsythe streets, extending north from the western end of the Manhattan Bridge to connect with the southern terminus of Second Avenue at Houston Street. This strip is plainly visible in the cover picture. Two drawings are reproduced here; one showing the proposed parkway and depressed roadway and the type of buildings that might be constructed along it under the regulations proposed by the Regional Plan; the other including the proposed development of Second Avenue with rapid transit, depressed express highway, and arcaded sidewalks within the building lines.

THE DIFFICULTY OF THE SMALL LOT

One of the difficulties experienced in securing adequate restrictions as to height is to obtain the consent of the owners of small plots from 25 to 50 ft. wide by 100 ft. deep. If all the small lots in central and sub-central areas are pooled together, a five-story height is perhaps the best paying height. In the most congested areas it is the ground floor that is most valuable.

Experts agree that low buildings in valuable locations pay a much higher percentage of income than high buildings. Emphasis should be placed on the word "valuable." In locations where the values are low there is even less profit in over-building.

But there is another factor entering into the problem, which helps to justify the claim that it does not pay to erect low buildings. Taxes on a low building will amount to about half of the gross rent, because, for instance, land containing a \$300,000 building will be assessed at \$800,000. If changes in the zoning law requiring lower buildings were accompanied by readjustments in assessed valuation to correspond with lowered densities, it would be still more profitable to erect low buildings. In other words, if the owners of 25-ft. lots were restricted to a building height of five stories and their assessments were adjusted on that basis, they might be better off, on the whole, than they are now with the right to build much higher structures.

GENERAL CONSIDERATIONS

The economic adjustments that every one must make these days seem to be causing the owners of skyscrapers more serious financial difficulty than the owners of low buildings. One fact at least stands out, that in proportion as skyscrapers have increased traffic congestion and the dark space in buildings, they have increased the financial burdens of the community and disturbed property investments. The skyscraper is having the fight of its life to justify the enormous cost it helped to impose on New York City for transit facilities, because of traffic congestion.

Regional planners do not question the efficiency of the skyscraper; they want to make it more efficient. They do not deny its beauty, but they want more of the play of light on its face so as to magnify its beauty. They do not underestimate the importance of concentration, but maintain that the process has its limits and that these limits are prescribed by economic laws governing the extent, character, and cost of traffic movement.

In conclusion, I quote from Vol. 2 of the reports of the Regional Plan:

> We have said that the invention of steam power changed the character of transportation. But the enormous power of steam had to be governed and restrained or it would have become a mere agent of destruction. The conditions being created by the skyscraper, as described in the regional survey, show that the skyscraper, too, must be kept within proper limits that give the maximum of utility, without the danger of destroying the very machinery of business whose operating efficiency it has been designed to increase. In a constructive sense it is restrained under the technical skill of the architect, engineer, and builder, who know the safety limits of building height. In a civic sense it is restrained by zoning, but not to the degree necessary to prevent dangerous congestion and to give proper display to buildings.
>
> As an American invention it has proved to be an instrument of efficiency as well as noble in architecture. When, however, it is so crowded that it becomes part of a great mass of buildings that closes in and darkens streets, it is anti-American in spirit, for America was founded on those conceptions of spaciousness and liberty that are together part of the life of its people.
>
> * * * *
>
> Whatever its limitations, the skyscraper has made a remarkable contribution to the architecture of New York. It is likely to continue to do so. New mountain ranges of buildings will probably arise in a few places, wherever accessibility to them is provided on the scale necessary to make them economically sound, just as they have arisen in downtown Manhattan but ceased to extend widely beyond a small area, and as they have developed in recent years around the Grand Central and Pennsylvania terminals. These ranges will reach a point where congestion and excessive cost of providing new transit facilities will limit their extension. Lesser ranges of compact high building will occur wherever new terminals are created, or, as in Brooklyn, where an important civic and business center, with elaborate transit facilities, makes them possible. Outside of these peak areas there will be groups of buildings of lesser height massed along water fronts or other areas of special attraction, while a large proportion of Manhattan and a still larger proportion of the other boroughs will continue to have a comparatively low level of building.
>
> In the areas of the higher and lower mountain peaks there will be protruding towers as there are now, and throughout the city other towers of less height will occur, with sufficient regularity to make them landmarks. Perhaps further developments will give to the high-building areas an accidental grandeur we cannot now foresee. But the general appearance of these areas cannot undergo much change toward a settled skyline in which elements of design have entered into the general composition.

This report lays emphasis on the fact that, as an individual building, the skyscraper has still to be given the chance to show its greatest value. In an esthetic as well as in an economic sense, its problems remain to be solved.

HINTS THAT HELP

Today's Expedient—Tomorrow's Rule

The minutiae of everyday experience comprise a store of knowledge upon which we depend for growth as individuals and as a profession. This department, designed to contain practical or ingenious suggestions from young and old alike, should afford general pleasure not unmixed with profit.

Design of Continuous Frames Having Variable Moment of Inertia

By Thor Germundsson

Associate Member American Society of Civil Engineers
Chicago, Illinois

BECAUSE of the wide use of the moment-distribution method proposed by Hardy Cross, M. Am. Soc. C.E., it is desirable to provide coefficients that will eliminate all integration work from the design of haunched beams used in continuous frames. The accompanying chart gives such coefficients. They have been computed by transforming values applicable to a design method proposed by A. Strassner. The range of the computed coefficients has been materially enlarged and their usefulness improved by presenting the results in the form of diagrams rather than in tables.

The moment-distribution method, as described by its author, Professor Cross, in the May, 1930, issue of PROCEEDINGS, requires a determination of the fixed-end moments in the various spans of a continuous frame and a progressing distribution of the unbalanced moments until sufficient accuracy has been obtained. The following values must be determined prior to the distribution: (1) end-moments due to the loads on the members when the latter are assumed to be fully restrained at both ends; (2) stiffness factors, or the ratios determining the distribution at each joint of that part of the moment that is not balanced by a moment of opposite direction; (3) carry-over factors, or ratios determining the amount of moment produced at the far (fixed) end by an additional moment at the near end.

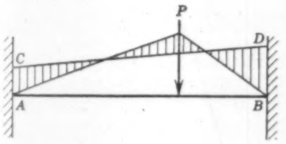


FIG. 1

A short description of how the coefficients may be determined by a general method, the column analogy, proposed by Professor Cross, in Bulletin 215, Engineering Experiment Station, University of Illinois, may be helpful.

For example, determine the fixed-end moments in the member *AB* in Fig. 1. Let *AB* be a fully restrained member with variable moment of inertia, *I*, supporting a concentrated load at point *P*. Let *APB* be the known moment curve obtained if *A* and *B* were hinged, and let *AC* and *BD* represent the unknown fixed-end moments. According to the column analogy principle, *AC* and *BD* can be determined as the extreme fiber stresses in a cross section subject to a normal force. The cross section is to extend from *A* to *B*, the width (perpendicular to *AB*) being proportional to the value of $\frac{1}{EI}$, *E* being the modulus of elasticity. The normal force, which is perpendicular to the plane of the paper, is to be equal to the total transformed moment area corresponding to the area *APB*, the moment at any point being transformed by dividing it by *EI*. This force is to be applied at the centroid of the transformed moment area. If either *E* or *I* varies between *A* and *B*, the determination of these areas, moments of inertia, and centroids involves integrations.

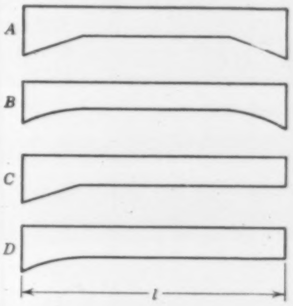


FIG. 2

A similar application of the column analogy principle may be used to determine also the stiffness factor and the carry-over factor.

The original integrations by Strassner are based upon the assumption that EI = constant × d^3, where *d* is the depth at the various points along the center line of the haunched members. The determination of the coefficients requires no additional assumptions, except those in common use for determination of extreme fiber stresses in column sections.

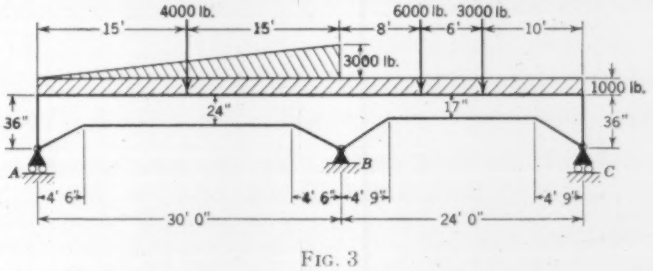


FIG. 3

Coefficients have been computed for the four cases *A* to *D* in Fig. 2. For each case, the values of *a* and *b* have been varied, *a* being the ratio of the length of the haunch to the span length, and *b* the ratio of minimum to maximum moment of inertia. The members in *A* and *B* are symmetrical, with *a* varying between 0.15 and 0.50; and those in *C* and *D* are unsymmetrical, with *a* varying between 0.20

FIXED-END MOMENTS, IN FOOT-POUNDS

	At *A* in *AB*		At *B* in *AB*	
1,000 lb. at point 6	0.140 × 4,000 × 30 =	17,000	0.140 × 4,000 × 30 =	17,000
4,000 lb. per ft., uniform load	0.092 × 1,000 × 30^2 =	83,000	0.092 × 1,000 × 30^2 =	83,000
Triangular load	0.070 × 1,500 × 30^2 =	94,000	0.114 × 1,500 × 30^2 =	154,000
Totals		194,000		254,000
	At *B* in *BC*		At *C* in *BC*	
6,000 lb. at Point 4	0.205 × 6,000 × 24 =	29,000	0.068 × 6,000 × 24 =	10,000
3,000 lb. at Point 7	0.112 × 3,000 × 24 =	8,000	0.187 × 3,000 × 24 =	13,000
1,000 lb. per ft., uniform load	0.100 × 1,000 × 24^2 =	58,000	0.100 × 1,000 × 24^2 =	58,000
Totals		95,000		81,000

and 0.10. The variation of the depth of the haunch is linear in A and C, and parabolic in B and D. The value of b varies in all four cases between 0.02 and 1.00.

The accompanying chart, Fig. 4, contains all the numerical values computed for Case A, as indicated. The end-moments for concentrated loads at Points 1 to 6 are to be determined from the values given in the first six rectangles, as follows: fixed-end moment at $\begin{Bmatrix}\text{left}\\ \text{right}\end{Bmatrix}$ support $= \dfrac{1}{1{,}000}\begin{Bmatrix}M_0\\ M_{12}\end{Bmatrix} \times Pl$, where P is the total concentrated load at the point considered. For the triangular load in Rectangle 7 and the uniform load (Rectangle 6, dotted) the total loads on the beam are used in the same formula. It should be noted that both K (far end of beam hinged) and K/C (far end of beam restrained) are to be multiplied by I/l in order to represent the stiffness of the members, I being the minimum value of the moment of inertia, and l the span length.

The use of the chart may be illustrated by a design example. In Fig. 3 is shown a two-span continuous beam with variable moment of inertia supporting various loads. Let the moment of inertia be proportional to the cube of the depths of the beams. Required to find the moment over the center support B.

The values of a are: for AB, $\dfrac{4.5}{30} = 0.15$; and for BC, $\dfrac{4.75}{24} = 0.20$. The values of b are: for AB, $\dfrac{(24)^3}{(36)} = 0.33$, say, 0.30; and for BC, $\dfrac{(17)^3}{(36)} = 0.105$, say, 0.10.

From Fig. 4 we find that the value of K for AB is 3.7, and for BC, 4.6. The stiffness is therefore (A and C being hinged): for AB, $3.7 \times \dfrac{24^3}{30}$; and for BC, $4.6 \times \dfrac{17^3}{24}$, the ratio of the former to the latter being 1.8. Hence, 64 per cent of the unbalanced moments at B are to be distributed to AB, and 36 per cent to BC. The carry-over factors (taken from the chart) are: for AB, 0.58; and for BC, 0.67.

We have now obtained from the chart all the values required prior to the distribution of moments, which follows:

	A	B		C
Distribution factors		0.64	0.36	
Carry-over factors	0.58		0.67	
Fixed-end moments	+194	−254	+ 95	−81
Balancing moments	−194	+102	+ 57	+81
Distributed moments	0	−113	+ 54	0
Balancing moments	0	+ 38	+ 21	0
Actual moments	0	−227	+227	0

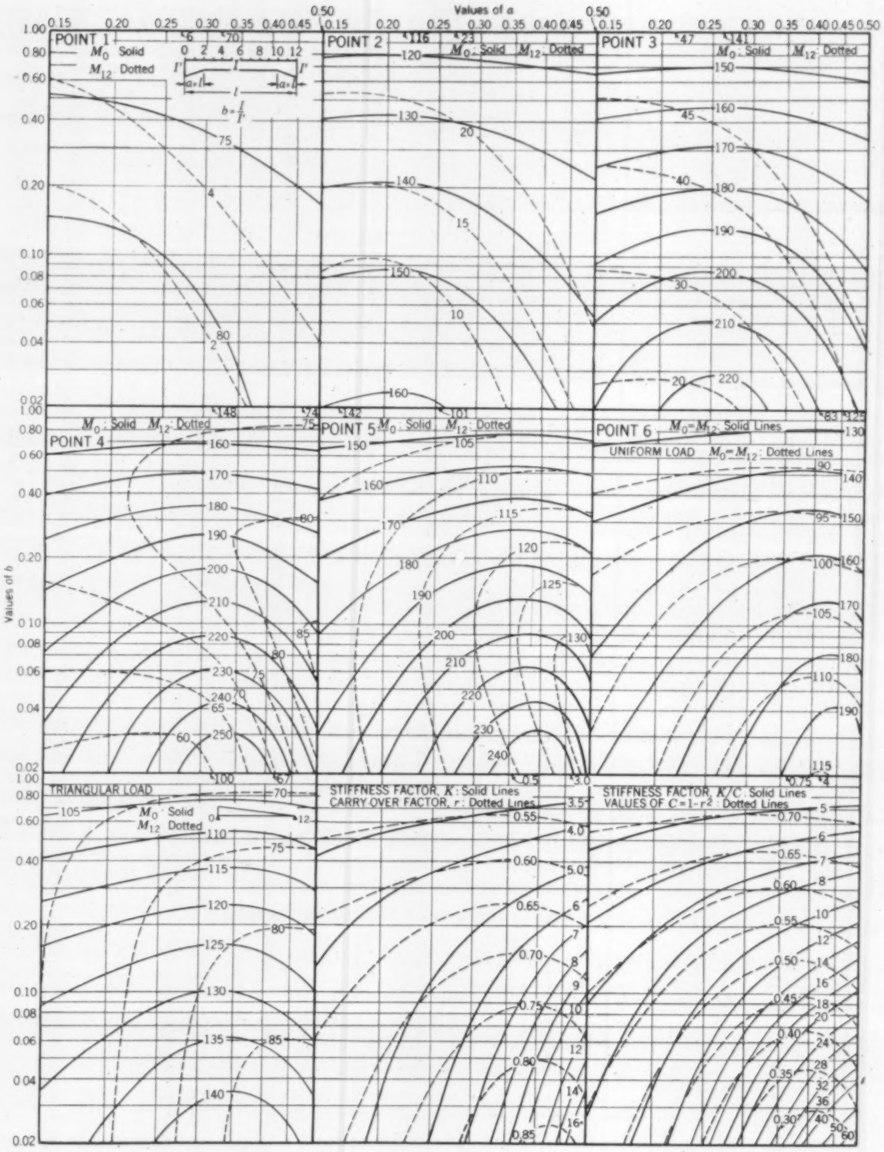


FIG. 4. COEFFICIENTS FOR DESIGN OF SYMMETRICAL BEAMS WITH STRAIGHT HAUNCHES AT BOTH ENDS

It is thus evident that the desired moment over the center support B is 227,000 ft-lb.

[For the convenience of structural designers and students, a complete set, including all four of the charts, on 8½ by 11-in. heavy paper, has been photolithographed and may be purchased from the Society, 33 West 39th Street, New York, N.Y. The price is $1.00 per set, and to members in any grade the usual reduction of 50 per cent is granted.]

Designing Footings by Charts

By PRESTON M. GEREN

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IN designing foundations which rest on compressible soil, it is important that the individual footings be proportioned so that settlement throughout the structure will be as nearly uniform as possible. In buildings designed for light occupancy, such as hotels, apartments, and office buildings, this is best accomplished by making the area of the individual footings proportional to the dead load only and, in buildings of heavier occupancy, by making it proportional to the dead load plus some percentage of the live load carried by the lowest column. The percentage of live load to be used in such cases depends on the class of occupancy and is largely a matter of judgment. In all cases, the soil pressure used in proportioning should be such that, for full load, the pressure under the footing having the highest percentage of live load will just equal the allowable bearing pressure of the soil on which it rests. As all footings must then be designed for the full computed loads, there is a wide variation in what may be called the "design soil pressures." For that reason, the use of the usual footing tables or footing design charts based on a constant soil pressure is not economical.

To facilitate the design of square footings proportioned as previously outlined, I have found it useful to prepare a series of charts based on the usual concrete strengths and on certain standardized relations between footing size and pier size. The concrete strength and the relation between footing size and pier size, are constant for each chart.

A typical chart, such as is shown in Fig. 1, is based on an ultimate concrete strength of 2,000 lb. per sq. in., the side of the pier being 32.5 per cent of the side of the footing. The allowable steel stress is 20,000 lb. per sq. in. and the allowable shearing stress in the concrete is 60 lb. per sq. in.

The curves are based on equations that are similar to those developed by A. R. Lord, M. Am. Soc. C.E., in a paper presented before the American Concrete Institute and later published as a *Handbook of Reinforced Concrete Construction* by the Portland Cement Association, the Concrete Reinforcing Institute, and the Rail Steel Bar Association.

These equations are as follows:

$$M = 0.5\,P\,b \ldots\ldots\ldots\ldots\ldots [1]$$

$$\frac{w}{v_c} = \frac{504\left(\frac{d}{b}\right)\left(2\frac{d}{b} + 0.325\right)}{1 - \left(2\frac{d}{b} + 0.325\right)^2} \ldots\ldots\ldots [2]$$

$$b = \sqrt{\frac{P}{w}} \ldots\ldots\ldots\ldots\ldots [3]$$

whence

$$d = \frac{d}{b}\sqrt{\frac{P}{w}} = \frac{\frac{d}{b}}{\sqrt{w}}\,\sqrt{P} \ldots\ldots\ldots [4]$$

$$A_s = \frac{0.042\,P}{17{,}500\,\frac{d}{b}} \ldots\ldots\ldots\ldots [5]$$

For bond, using deformed bars with hooked ends:

$$\text{Bar diameter} = 0.0675\,b \ldots\ldots\ldots [6]$$

In these equations the notations used are as follows:

M = bending moment in inch-pounds
A_s = steel area in one direction in square inches

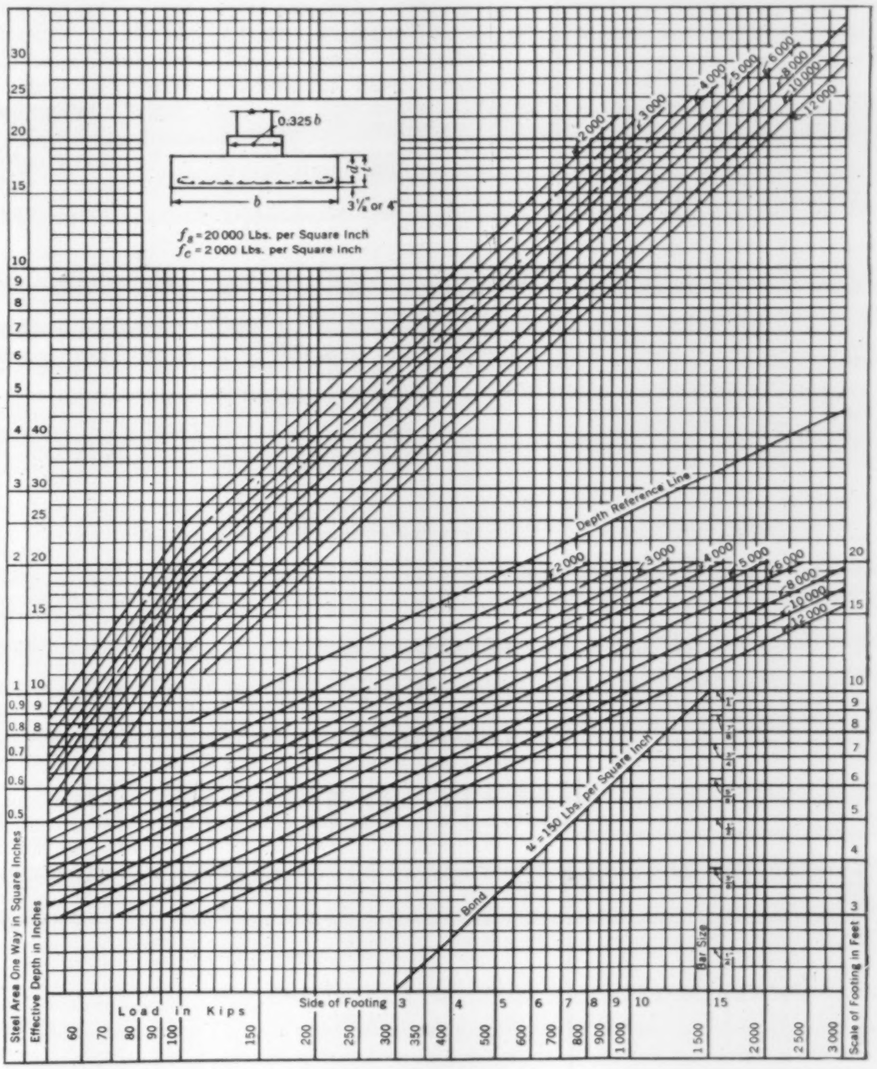


Fig. 1. Chart for the Design of Footings

b = side of square footing in feet
d = effective depth of footing in feet
P = load at top of footing in pounds
w = soil pressure in pounds per square feet
v_c = unit shearing stress on concrete at the critical section in pounds per square inch

Table I. Calculations for Foundation Design

(1) w	(2) $\sqrt{w}$	(3) $\frac{d}{b}$	(4) $\frac{\frac{d}{b}}{\sqrt{w}}$	(5) $\frac{d}{b}$ (adjusted)
2,000	44.72	0.0945	0.00211	0.098
2,500	50.00	0.1080	0.00216	0.110
3,000	54.77	0.1196	0.00218	0.121
3,500	59.16	0.1300	0.00219	0.130
4,000	*63.25*	*0.1393*	*0.00220*	*0.139*
4,500	67.08	0.1475	0.00219	0.148
5,000	70.71	0.1551	0.00219	0.156
5,500	74.16	0.1622	0.00218	0.163
6,000	77.46	0.1682	0.00218	0.170
8,000	89.44	0.1895	0.00212	0.197
10,000	100.00	0.2055	0.00206	0.220
12,000	109.55	0.2185	0.00200	0.241

The values in Col. 3 of Table I were computed from Equation 2. The values in Col. 4 are the first terms of the right side of Equation 4 for the values of w listed.

An inspection of Col. 4 reveals that, for any value of P, the effective depth of the footing becomes maximum for a soil pressure of 4,000 lb. per sq. ft. It is also evident that the total variation in depth from minimum to maximum for any load is so small that, for practical purposes, the maximum value may be used for all soil pressures, provided the values of $\frac{d}{b}$ used in Equation 5 are adjusted proportionately. These adjusted values are tabulated in Col. 5.

The "depth reference" line on the chart, Fig. 1, was plotted from Equation 4, using the maximum value of $\frac{\frac{d}{b}}{\sqrt{w}}$, from Table I. A minimum effective depth of $8^1/_2$ in., corresponding to a total depth of 1 ft., was adopted.

The lower group of "soil pressure" curves were plotted from Equation 3. The upper group were plotted from Equation 5, using the adjusted values of $\frac{d}{b}$ from Col. 5 of Table I. For the minimum depth of $8^1/_2$ in., the value $d = \frac{8.5}{12}$ was substituted, the equation becoming:

$$A_s = \frac{504\,P\,b}{148{,}750} \quad \ldots\ldots\ldots\ldots [7]$$

The method of using the chart may be described as follows. Having determined the size of the square footing required, enter the bottom of the chart with the "design load" (total load at the top of the footing, including weight of pier). At the intersection with the footing size read the "design pressure," interpolating intermediate values. At the intersection with the "depth reference line" read the effective depth from the scale at the left. At the intersection, in the upper group, with the "design pressure" read the required area of steel (one direction) from the scale on the left. From the inset "bond chart," determine the maximum size of bar that may be used.

As all the values of d have been determined from the maximum value of $\frac{\frac{d}{b}}{\sqrt{w}}$, the value of v_c will be less than the assumed allowable value except when the "design pressure" is 4,000 lb. per sq. ft.

For those who prefer an all slide-rule method, Equations 3 and 5 may be extended as follows:

For $\frac{\frac{d}{b}}{\sqrt{w}} = 0.0022$, converting d to inches,

$$d = 0.0264\,\sqrt{P} \quad \ldots\ldots\ldots\ldots [8]$$

$$A_s = 0.00109\,\sqrt{P\,b} \quad \ldots\ldots\ldots\ldots [9]$$

Similar equations may be developed and similar charts prepared for other strengths of concrete and for other ratios of pier size to base.

OUR READERS SAY—

In comment on Papers, Society Affairs, and Related Professional Interests

Notable Arch Tests in Progress

DEAR SIR: On page 493 of the August issue, there is a photograph of the experimental three-span arch being tested in the Materials Testing Laboratory at the University of Illinois. This shows the set-up of what to me has been an exceedingly interesting series of tests. Perhaps a little more complete statement of what is being done will be of value to readers.

The entire structure rests upon scales which permit the exact weighing of the horizontal and vertical components of the reactions at both abutments and both pier bases. The structure is also equipped with precision instruments which perform the following functions: (1) restrain the abutments and pier bases so as to prevent rotation and vertical or horizontal translation, in the case of tests for which the terminals are fixed; (2) rotate the abutments and pier bases through a predetermined angle or give them a predetermined horizontal or vertical translation, in the case of tests to determine the effect of spread of abutments or settlement of foundations; and (3) determine the deflection of points on the arch rib due either to load or to movement of the abutments and pier bases.

The tests were made during June, July, and August. These tests included:

Series 1. Determination of the influence ordinates for reactions by applying a unit load of one ton, successively, at each of the 24 load points.

Series 2. Experimental determination of the elastic constants of the various spans.

Series 3. Determination of the deflection of the load points due to unit movement of the abutments and pier bases.

Series 4. Determination of the reactions at the abutments and piers due to the design load, a combination of dead and live load.

Series 5. Test to failure.

The apparatus indicating the position of the bases of the piers was applied at points 10, 15, and 20 ft. below the springing of the arch so that the pier could be fixed at these points. Fixing a point in the pier 10 ft. below the springing is equivalent to having a structure with 10-ft. piers, whereas fixing the points 20 ft. below the springing is equivalent to having a structure with 20-ft. piers. Thus having the instruments indicating the position of the pier base located at points 10, 15, and 20 ft. below the springing of the arch, made the one structure shown in the photograph equivalent to three structures varying in pier height but otherwise identical. Of the five series of tests enumerated, all but Series 2 and 5 were made at all three pier heights. Series 2 does not involve the height of the pier, and of course only one test to failure was possible. The apparatus, all of which was designed and built specially for this series of tests, functioned very nicely, and the results indicate that a very high degree of precision was obtained.

Forms have already been erected for a structure similar to the one shown in the photograph in your August issue except that each span will comprise a rib with a deck consisting of spandrel columns and a floor slab. This structure will be poured about the middle of September, and in the late fall, will be subjected to tests similar to those previously described.

I was assisted in planning these tests by G. E. Beggs, A. C. Janni, and C. T. Morris, Members Am. Soc. C.E., and E. H. Harder, Assoc. M. Am. Soc. C.E., all members of the Society's Committee on Concrete and Reinforced Concrete Arches; and by E. F. Kelley, M. Am. Soc. C.E., and A. L. Gemeny, both of the U.S. Bureau of Public Roads.

W. M. Wilson, M. Am. Soc. C.E.
Research Professor of Structural Engineering
University of Illinois

Urbana, Ill.
August 10, 1932

Control of Water Growths

To the Editor: In connection with Mr. Debler's paper, in the September issue, I should like to comment on aquatic growth, its effect on carrying capacities, and control measures. A part of the Eastern Section Irrigation System of the Canadian Pacific Railway Company near Brooks, Alberta, consists of the Lake Newell Storage Reservoir, supplied by a diverting canal from the Bow River and in turn supplying a block of irrigated land. The surface area of the reservoir varies from 8,400 acres at sill elevation of the outlet canal, at 2,465 ft. above sea level, to 16,000 acres at full supply level 20 ft. higher. The location is in the short-grass country, where the site was prairie sod and the formation glacial and largely clay. Careful studies indicate no measurable seepage.

Water was first turned into Lake Newell in 1914. As early as 1918 aquatic growth was noted in the irrigation canals, and by 1921 a serious problem had developed. Control measures, which were started in 1922, have been continued, as required, to date. Lake Newell is considered the primary source of the aquatic growth, as very little has developed in those parts of the system which are served by direct diversion from the river and which carry considerable river and local silt. The seeding of the lake is presumed to have been by waterfowl, and the spread through the canals probably by floating seed, bulbs, and waterfowl.

A study and report were made in 1925. Twelve important forms of water weeds (vascular plants with root stem and leaf) were enumerated. Of these, five (all of the Potamogeton family) caused most of the choking in canals and ditches. Algal flora, abundant both in variety and quantity, were also found, but this has not been a source of serious trouble.

Removing Aquatic Growth at Aqueduct Intake, July 1922

The weeds have a perennial bulb or root and grow at a tremendous rate in hot clear weather when the irrigation demand is high, sometimes extending through a depth of 8 ft. of water and in such thick beds as to almost completely choke canals within a few weeks. For example, consider the specific measurements made in 1922: on June 30 a flow of 660 sec-ft., indicated by the rating curve, was actually reduced to 435 sec-ft., or 34 per cent. Dragging started, and on July 14 the reduction was from 370 to 340 sec-ft., or 8 per cent. Choking again developed rapidly, with a maximum reduction on September 20 from 360 to 70 sec-ft., or 81 per cent. Thereafter, it diminished as a result of cooler or cloudy weather, or of the fact that the weeds settled to the bottom after the blooming period. Their growth appears to be influenced by sunlight rather than by water temperatures, and is favored by the clear water of the lake and the low gradient canals leading from it, in which there is almost no silt.

Study indicated that poisoning the weeds was not feasible. Consideration was given to the use of suspensions of finely powdered material, such as coal or clay, to shut off the light, but the cost appeared too high for the uncertainties of an experiment. Various kinds of drags were tried, including harrows, discs, chains, and cable. It is now standard practice to drag as required, with chains weighing from 15 to 20 lb. per lin. ft. looped across the water section and connected to the teams on the banks by cable. Swivels are required, as the chain rolls, and this rolling gives it self-cleaning properties. The control is reasonably effective at moderate costs of from $1 to $2 per mile for each dragging, but does not eradicate the growth. The number of draggings per season has varied from 5 to 25.

Dragging Tilley Canal with Harrows to Destroy Weeds, August 26, 1921

In the preparation of this discussion, I have been assisted by A. Griffin, M. Am. Soc. C.E., and F. G. Cross, Chief Engineer and Assistant Superintendent, respectively, Operation and Maintenance, Department of Natural Resources, Canadian Pacific Railway.

S. G. Porter, M. Am. Soc. C.E.
Manager, Department of Natural Resources, Canadian Pacific Railway Company

Calgary, Alberta
September 7, 1932

Crop Demand and Canal Capacities

To the Editor: The paper by Mr. Debler, in the September issue, is a good résumé of capacity in canal design and operation. Its title might well be "canal capacity in terms of crop demand," for the canal capacity is the dependent variable.

Granted that the capacity must be based on the peak demand, with a margin for safety, the solution of the problem should begin with a study of water requirements at the point of use. The author's discussion of water losses overshadows this more important factor. Before the Coolidge Dam project was begun, I made a monthly use-of-water chart for the valley which now receives its supply from the Coolidge Reservoir. It was assumed that one third of the acreage would be in alfalfa, one third in cotton, and one third double-cropped, as with wheat followed by sorghum, or winter lettuce followed by cantaloupes. The amount of water required monthly for each crop is known within reasonable limits, and the chart was prepared to show the average (Fig. 1).

In other cases assumptions may be based on a project with different soil and climate, and allowances will have to be made for length of growing season, humidity, and other factors. Determination of water requirements in the field is slow and costly.

Assumptions as to acreage of each crop should be based on conditions expected after full settlement. As an example of what may happen temporarily, the engineering structures of the Roosevelt Irrigation District were completed, permitting water delivery, on April 11, 1928. Within twelve months the entire acreage of 40,000 acres was calling for water, but it was in large holdings and almost wholly planted with cotton. The peak demands were abnormally high and necessitated the development of groundwater supplies within the irrigated area. With diversification of crops the auxiliary supply will not be needed. Irrigated areas in the semi-tropical belt are not always free from the disadvantage that the author ascribes to those in the Northern states. Even on old projects crop programs are changed.

Water storage in the soil, which is practicable if the soil is deep and retentive, is practiced to a considerable extent in parts of Arizona.

Ordinarily it would be assumed that the losses listed by the author should be reduced to the minimum practicable. For every irrigation rule, however, there are exceptions. On the Yuma mesa it is desirable to apply frequent irrigation after June 20, for the purpose of increasing the evaporation loss and thus lowering the soil temperature. In several areas in Arizona it is desirable to maintain a downward movement of soil water to prevent the concentration of alkali salts.

Compilations of diversions, losses, and deliveries should be read with circumspection. For example, analyses of canal operation for 22 U.S. Bureau of Reclamation projects were published in the *Reclamation Record* of September 1918. For the Salt River project the average use of water for the preceding six years was shown to be 2.89 ft. per year. A long list of measurements by hydrographers sent out in response to protests showed a general over-delivery of 20 per cent. For the cases where no protests were made the over-delivery may have been 30 or 40 per cent. The deductions from the records were far from correct. It caused me embarrassment because promoters felt that I advised a water duty which was much too low.

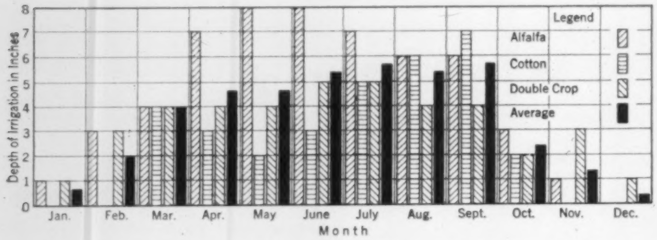


FIG. 1. AMOUNT OF WATER REQUIRED IN IRRIGATION OF VARIOUS CROPS

The author has referred to the efforts of legislatures to limit the diversions and the resulting difficulties. The power to fix such limitations and to enforce them should repose in state water commissions or the state engineer, and uniformity throughout a state is not warranted. No other body of men is so competent to deal justly with the irrigator and, at the same time, to protect the public, whose chief interest lies in maximum use and minimum waste.

G. E. P. SMITH, M. Am. Soc. C.E.
Professor of Irrigation Engineering
University of Arizona

Tucson, Ariz.
September 6, 1932

Practical Nomographic Chart for Field Work

TO THE EDITOR: In Mr. Shepard's nomographic chart for steel tapes, described in the July issue, he has provided a practical instrument that enables the man in the field to solve an intricate expression with all needful accuracy and at almost no expense in time.

The cartographer's greatest interest will perhaps center on Scales E, F, and G. Possibly through a desire to shorten his article, Mr. Shepard presents the reasons for making the chart and explains its construction, without emphasizing such questions as the adjustment of scale factors and the relative positions of scales. Without careful attention to these, however, he could not have fulfilled the requirements for a field chart covering certain limits, readable with a certain degree of precision, and giving suitably definite intersections.

This is particularly true in respect to Scale F. As there is virtually unlimited choice in placing the zeros of E and G, the curvature as well as the absolute length of F may vary widely, and can be properly fixed only after consideration of all requirements. In view of the fact that the chart is designed for the field, and must therefore be limited in size, it undoubtedly is well adapted to its purpose. Mr. Shepard's analysis enables one to construct without difficulty a similar chart of any desired size and range.

ROBERT C. STRACHAN, M. Am. Soc. C.E.
Consulting Engineer

New York, N.Y.
July 19, 1932

Rigid-Frame Bridges in Westchester County

TO THE EDITOR: Referring to the paper by J. W. Beretta on "Rigid Frames and Continuous Concrete Spans," in the September issue, this type of construction is not new, having been used in Europe for building frames and other purposes for a great many years. It has also been used to some extent for the same purposes in this country.

It was only ten years ago, however, that this type of construction was adapted to bridges by varying the sections of the principal members and devising design methods to take such variation into account. Mr. Beretta was correct in giving the credit for this adaptation to A. G. Hayden, M. Am. Soc. C. E., Designing Engineer for the Westchester County Park Commission. Mr. Hayden worked this design out in 1921 after a thorough analysis and tests of models had been made by consulting engineers.

The first bridge of this type was built on the Bronx River Parkway at Bronxville, N.Y., in 1922. Since that time, 84 more have been built in Westchester County, principally by the Westchester County Park Commission. Of these, 77 are of reinforced concrete construction, generally with stone facing, and 7 are of steel construction. In all the rest of the world, so far as our records show, only about 85 of these rigid-frame bridges have been built, mostly since 1927. It is probable that there are a few others of which we have no knowledge.

In bringing this type of construction again to the attention of engineers, Mr. Beretta is to be commended. It deserves more consideration than it has received up to the present time for several reasons. The first is in the matter of economy in the use of materials and, therefore, in the cost of the structures. Another reason is that this form of construction lends itself to pleasing architectural treatment better than the usual form of bridge design, such as trusses, steel plate girders, and reinforced concrete girders. There is another very decided advantage in that the thickness of the overhead structure can be materially cut down, thus creating a saving in the cost of the approach fill, particularly in flat country where long approaches must be built.

A RIGID-FRAME BRIDGE STRUCTURE OF REINFORCED CONCRETE Over the Hutchinson River Parkway, Mount Vernon, N.Y.

It is probable that the rigid frame has not been more generally used because of the rather complicated mathematics involved in

A Steel Rigid-Frame Bridge, With 117-Ft. Span

Carrying Palmer Avenue Over Central Avenue, Yonkers, N.Y.

the design. However, in the practice of the Westchester County Park Commission these mathematics have been reduced to a minimum, and the methods used are fully described by Mr. Hayden in his book, *The Rigid-Frame Bridge,* and in various Society publications.

The accompanying illustrations indicate the results that can be secured in the use of the rigid frame for short span bridges.

L. G. Holleran, M. Am. Soc. C.E.
Deputy Chief Engineer, Westchester County Park Commission

Crestwood, N.Y.
September 9, 1932

Early Rigid-Frame Bridges

To the Editor: The paper by J. W. Beretta, in the September issue, is a very interesting presentation of some recently constructed rigid-frame concrete bridges. The attention which has of late years been directed to this type of structure in this country, may lead to the impression that it is an entirely new development. This does not happen to be the case. The rigid-frame concrete bridge was used in Europe as early as 1904. It will be remembered that rational theories of reinforced-concrete action did not appear until about that time. The location, designer, and date of construction of some of these early bridges are given in the following tabulation:

Location	Designer	Date
Poprad, Hungary	Zielinsky	Before 1908
Markersbach, Saxony	Hennebique	Before 1904
Dobeln, Saxony	Royal Saxon Railways	1904
Dennheritz, Saxony	Royal Saxon Railways	1905
Meerane, Saxony	Royal Saxon Railways	1906
Vilsbiburg, Bavaria	Dickerhoff and Widmann	Before 1911
Blackburn, England	Patent Indented Steel Bar Company, Ltd.	Before 1911
Aach, Switzerland	Swiss Federal Railways	1907

In addition to these, the Swiss Federal Railways made extensive use of the rigid-frame concrete bridge for crossing structures during the reconstruction of the Winterthur-St. Gallen line, in the period, 1910–1914. Some of the bridges erected at that time are: Eschlikon (1911), Moosburg (1912), Gloten (1912), Jfwil (1912), Guntershausen (1912), Haldenhof (1913), and others. The first seven bridges in the tabulation are single-span structures. Most of the Swiss examples cited have two or more spans.

Referring to the Martinez Street Bridge described by Mr. Beretta, it would be of interest to know why the reaction lines, shown in his Fig. 3, for the dead load of 176 kips and the combined dead and live load of 210 kips are vertical and pass almost through the axis of the knee. This does not appear reconcilable with the characteristics of rigid frames nor, more specifically, with the knee moments given in Mr. Beretta's Fig. 2.

A. A. Brielmaier, Jun. Am. Soc. C.E.

Cleveland, Ohio
September 3, 1932

Standard Specifications for Continuous Frame Bridges

Dear Sir: In his discussion of "Rigid Frames and Continuous Concrete Spans" for "workhorse" bridges, in the September issue, Mr. Beretta fails to mention the opposition of "standard specifications" to such designs. A description of an actual case in an "up-state" county job may therefore be of interest.

In response to an invitation to submit a proposal and plan for a highway crossing of a creek, a contractor submitted a two-span continuous girder bridge with concrete deck and balustrades, rigidly connected to ribbed abutments. The bid, which was less than $30,000, was the lowest bid received. The next bid for a two-span arch with massive abutments but no road surface was almost $40,000.

The low bid was declared irregular because the standard specification required gravity abutments; and since the invitation for proposals asked for a bridge and advised that another contract would be let for the construction of the road, the road surface over the bridge and the fill on the arch barrels were included in the road contract. Perhaps the standard specification has been changed, and the $10,000 difference may be of greater value now than it was a few years ago, yet the average highway "workhorse" bridge is far from an economical design. Of course, the bridges built by Arthur G. Hayden, M. Am. Soc. C.E., and others on both the Westchester and Long Island Parkway Commission jobs are exceptions.

In tabulating resultants on the foundation for the Martinez Street Bridge (Mr. Beretta's Fig. 3), I believe that an investigation of deformations would have shown a displacement of the abutment toward the backfill when full live load is assumed. In that case the earth pressure should be taken as greater than the active earth pressure, since it is closer to the passive resistance of the backfill. In either case, a vertical component from earth pressure will exist at the abutment and, together with the weight of the fill wedged in the narrow spaces between ribs, should be considered in the foundation design.

Jacob Feld, Assoc. M. Am. Soc. C.E.
Consulting Engineer

New York, N.Y.
September 7, 1932

Notes on Pile Resistance

To the Editor: As Mr. Miller states in his article in the May issue, the design of pile foundations is not an exact science. Piling companies have not done much beyond standardizing processes for their respective types of piles. Competition is keen and does not permit generous expenditures for pure research. Soil conditions are so variable that each particular case would remain an individual problem in spite of research, but the piling and foundation companies have a great mass of information in the aggregate which, if properly analyzed and prepared, should be of inestimable value to the profession.

The pedestal undoubtedly adds bearing value to the pile. In addition, in soils where the skin friction of a steel, or any other smooth pile might be negligible, the outward spread of the concrete under compression expands the shaft against the surrounding soil, adding a skin friction value. This is shown in the photographs of exposed pedestal piles which illustrate Mr. Miller's paper.

As an instance of skin friction value, I drove piles for a structure some years ago. The design called for an anchorage against lifting. Rods with gripping heads welded on them were cast in these piles, the upper ends being formed into a hook. Pull tests were made with the pile-driving engine, the pull being continued until the rod broke. The rods had been previously tested in the laboratory for their ultimate strength, and from this the force of the pull was determined to be over 40 tons. Notwithstanding this fact, the piles which were only 30 days old stayed permanently in place, in filled ground partly aqueous, proving that this type of cast-in-place pile, with its large area of rough exterior surface, has a high skin-friction value.

Although pile driving is not an exact science, adequacy and safety may be assured by infinite care. Preliminary tests will indicate the best type of pile. The *Engineering-News* formula will be a rough index of the permissible load, and can and should be supplemented by actual load tests. Lastly, rigid inspection during every minute of the driving will ensure proper execution of the work. In fact this might aptly be termed the crux of the problem. I regard it as most important because good results depend upon good workmanship which, in turn, is controlled by inspection.

Another word of caution might be added. Sometimes a condition exists where a stratum of clay or other tenacious material overlies or floats on a poor aqueous soil. If the water should seep away, the bearing value of this lower zone might be entirely gone. The good layer has a limited bearing value and a resistance of its own, but it is not reliable because of the conditions above and beneath it. Preliminary borings would expose such a condition, and the cautious engineer would insist that the pile be driven through this stratum regardless of the hard driving.

Much stress has been laid on a common belief as to what occurs to the shaft of a new concrete pile without a casing when another pile is driven next to it. Many persons with whom I have discussed this question set up hypothetical assumptions, but my observation of thousands of these piles driven in all kinds of soils does not bear them out. Many of these piles which are heavily loaded—up to 60 and 70 tons—have been dug up for inspection. The findings absolutely refute the theoretical assumption that fresh piles are distorted by driving adjacent piles.

L. S. STILES, M. Am. Soc. C.E.
Construction Engineer
The Brooklyn Union Gas Company

Brooklyn, N.Y.
August 7, 1932

National Park Projects Present Engineering Difficulties

TO THE EDITOR: Within our National Parks engineering problems must be handled by those who fully appreciate the purpose of the parks. In his article in the September issue, Mr. Kittredge has stated that so-called improvements are not really improvements since mere man cannot improve the work of Nature within these areas. This is very true. These regions of outstanding scenic or scientific interest must, however, be made accessible to visitors, and sanitary quarters must be provided for the accommodation of guests. These simple requirements involve a myriad of problems—many of them not easy to solve without violating a cardinal rule of leaving the parks unimpaired for the enjoyment of future generations.

The engineer assigned to these tasks must at once become "park-minded." Otherwise, he has no place in the work because he may cause permanent and irremediable damage. The closely related problems of construction and operation are equally fascinating to the park-minded. The marked success achieved by members of the engineering profession who have left problems of construction for those of administration indicates the closeness of the relationship. Among these members are Superintendent Toll, of Yellowstone National Park, and M. R. Tillotson, Assoc. M. Am. Soc. C.E., Superintendent of Grand Canyon National Park.

A really satisfactory definition of National Parks has not yet been written. Although they are often referred to as the "Nation's playgrounds," this is an ill-advised definition, as they are too remote to be reached by masses of people. This is perhaps fortunate because, as Mr. Kittredge has stated, many of them contain scenic, scientific, and historic treasures so precious that numerous visitors might destroy them. They must not be confused with the parks that our principal cities have provided. Their purpose is entirely different, as is the motive impelling their being set aside for preservation unimpaired for the use of future generations.

This distinction is too often not appreciated. We are threatened with a deluge of what I like to call "chamber of commerce national parks," promoted and urged by local communities which claim that their particular state or locality has no National Park while other regions have been more favored. It is not realized that these other localities have been favored by Nature and not otherwise. The inclusion of indiscriminate areas of mediocre scenic or scientific value within our system of National Parks would be certain, in the long run, to undermine and destroy the whole system.

The results obtained by the engineering staff of the National Park Service, working in close cooperation with the landscape department, are very praiseworthy. These engineers do not plan great structures to attract public attention. On the contrary, they are very careful to plan their projects so as not to attract attention and thus distract from the great works of Nature. The public, in using roads and trails, often does not realize the care that has gone into locating them, so that advantageous viewpoints may be reached by moderate grades that do not mar the landscape. Neither does this public appreciate the problems of water supply and sanitation which have been solved for its comfort and health. The men who direct these operations must, and do, receive their reward in the satisfaction that comes from work well done.

Problems of construction policy are ever pressing for solution in the National Parks. This has been particularly true in recent years of extremely active road construction. Fortunately, wise administration by former Director Stephen T. Mather and by his successor, Horace M. Albright, and their assistants has done much to save the National Parks from despoliation by unwise road development.

WALTER L. HUBER, M. Am. Soc. C.E.
Civil Engineer

San Francisco, Calif.
September 8, 1932

Development of Columbia River Projects

TO THE EDITOR: I was greatly interested in Colonel Robins' paper on the Columbia River in the September issue. As project engineer in connection with the investigation of The Dalles and Cascade Rapids projects on the Columbia River in 1914, I formed rather definite opinions regarding the commercial feasibility of developing this river. The general facts are as follows: (1) dependent irrigation is expensive; (2) power projects are too large to fit into the program of a gradually developing market; and (3) there is little demand for navigation.

Most chemical and metallurgical industries and all general manufacturing are far more dependent upon cheap and abundant sources of raw material and cheap transportation than upon cheap power. Except in the case of a very few industries, the cost of power is not the controlling cost of production. Such items as the supply of raw material, cost of labor and real estate, transportation, and taxes are nearly always the controlling factors.

The Pacific Northwest is very weak in its supply of raw materials. General manufacturing, with few exceptions, must depend upon distant sources of raw material and distant markets, such as iron from Cuba or China, coal from Manchuria or Alaska, bauxite from British Guinea or Africa, copper from Chile, and limestone from tidewater areas, if available. With distant sources of raw material and distant markets, the problem of transportation becomes doubly important, probably overshadowing all other handicaps to the development of Columbia River power. Moreover, this means that the power projects which are, or can be, made accessible to deep-draft ocean navigation have the best opportunity for development.

In my opinion, only two of the Columbia power projects—the Warrendale and The Dalles projects—can figure very largely in initial industrial growth, and that is because of their relatively close proximity to the sea. Power, raw materials, and markets must be brought together by cheap transportation, as the fewer the handicaps the better the prospects of commercial success. Practically all materials must come from distant points in ocean-going ships. To transship at Vancouver to river boats, or to transmit power to Vancouver or Portland, is a handicap that would in many cases cause failure.

There is one way in which the Federal Government can reasonably be called upon to help industrialize this area, and that is by the exercise of its normal function in providing navigation facilities. If not already acquainted with the facts, the Federal Government, through the U.S. Engineers, should study thoroughly the possibilities of providing and maintaining a deep-draft channel to the

Warrendale site, a lock for ocean-going vessels through the Warrendale Dam, and a deep-draft channel from there to The Dalles. If practicable, these developments would enable the use of these sites by chemical and metallurgical industries whose markets are world markets, whose raw materials come from distant sources, and for whom power cost comes nearer to being a controlling factor in the cost of production than it does in any other industry. In some cases, such industries cannot bear the cost of high-tension transmission, and they usually cannot seek locations at remote power sites unless these sites are on tidewater.

From a personal knowledge of the Warrendale and The Dalles sites, I realize the unprecedented engineering problems that must be solved, especially at the latter site. If foundation conditions have not been thoroughly investigated, a study of them should be financed as a prerequisite to any further approach to the problem. However, the real problem of developing the Columbia River is not the technical one, no matter how unprecedented, but the problem of finding sufficiently cheap raw materials, and transportation facilities to permit entering world markets.

L. F. Harza, M. Am. Soc. C.E.
Consulting Engineer

Highland Park, Ill.
September 7, 1932

Revised Economies of Single-Lane Pavements

Dear Sir: The discussion on single-lane pavement for local roads by Mr. Catton, in the August issue, is an interesting attempt to prove the economic worth of a certain type of construction by means of a formula which appears quite plausible until subjected to careful analysis, after which the fallacy of the reasoning becomes evident.

The formula itself is purely empirical and at best represents the opinion of only a small group. Its accuracy was immediately questioned by two of the ablest highway engineers in the country, when it was first presented before the Highway Research Board in the form of a committee report to justify the large expenditures required to take care of heavy truck traffic on certain trunk lines.

Assuming, however, that the formula has some merit and that there may be an honest difference of opinion in regard thereto, the use of certain figures is clearly unwarranted:

1. The difference in cost of operation of motor vehicles over untreated graded surfaces and high type surfaces is not on an average $1\,^3/_4$ cents per mile; in fact it does not exceed 1 cent per mile. The test data referred to are many years old and were obtained under conditions that are not representative of those found today either for roadbed or motor vehicle.

2. The studies referred to, in regard to time required for overtaking or passing opposing vehicles, were made on multiple-lane pavements of equal supporting value in all lanes. The conclusions arrived at under such conditions are certainly not to be taken bodily as representing the situation on a single-lane pavement where the passing is made on an earth shoulder.

3. The majority of concrete pavements in the United States are less than ten years old. There is certainly no quality evident in those of greater age to indicate that such a type of surface has a life expectancy of twenty-five years with but nominal maintenance. On the contrary, careful examination of such surfaces inescapably leads to the conclusion that fifteen years at most may be counted on before substantial maintenance is required.

Making substitution of proper revised figures in the formula will give a result somewhere between 350 and 500 vehicles per day as the minimum required traffic to justify the expenditures cited.

Even assuming all the liberal estimates to be correct, the community building such a single-lane type has only one half of a road with all the hazards naturally inherent in such restricted design. With the further necessary expenditures, the single-lane type actually costs not one-half but nearly three-quarters as much as a full width surface.

This all leads inescapably to the question, if the same costs are incident alike for grading and draining half-width concrete pavement as for a full-width one, is there not some less expensive form of surface that can be laid for the full width of such prepared subgrades which will be safe and convenient at all times? During the past few years, tens of thousands of miles of bituminous surfaces of the low-cost type have been constructed for widths of from 16 to 28 ft. at costs materially less than those required for a single 10-ft. lane of concrete. Sufficient time has elapsed to prove their economy beyond all question. It is hardly likely that the taxpayer will be content with half a loaf when his desire for good roads can be satisfied to the full with substantially lower expenditure.

Bernard E. Gray
Highway Engineer

New York, N. Y.
August 27, 1932

Development of Steel-Pile Foundations

To the Editor: The paper by Messrs. Mason and Ogle, in the September issue, covers a subject not often referred to in engineering discussions; in fact, practically no technical data pertaining to steel H-piles have ever been published. The authors have shown that this type of piling has been used in the state of Nebraska for over thirty years. They have failed to state, however, that over 10,000 bridges have been constructed on steel H-pile foundations in the Middle West during this time. The Platte River is subject to heavy scour conditions during floods, and the authors have demonstrated that an economical solution to the problem of building bridge structures on safe foundations has been found by the use of steel H-piles, mat protection, and jetties.

The early type of steel-pile foundations consisted of I-beam piling, which was used until 1908, when wide-flanged sections and H-columns were placed on the market and the I-beam section for piles was abandoned in favor of the H-section. Real development in the use of steel H-piles began about 1912, at which time the state of Nebraska began to develop its standard plans for this type of bridge foundation.

The foundation material found under the Platte River consists of deposits of heavy gravel, and the driving of piles into this type of material has always been difficult. It was found that steel piles permitted driving into this material with very little damage to the piles, and that the type also has very high load-carrying capacity. Steel piles usually wrinkle at the top, as stated by the authors, but recent installations have indicated that they will wrinkle several feet below the surface of the ground as well, when driven into hard gravel. This trouble has been overcome by the use of heavier sections and by regulating the fall of the hammer.

Bridge structures on steel-pile foundations have been built in 15 other states, and the piles have been driven in foundation material varying from soft clay to shale rock. Bridge foundations of this type have been used in the Western states since 1916, and the first structures were constructed for the U.S. Indian Service near Umatilla, Ore., in that year. During the last four years approximately 25 steel-pile bridges have been built in the state of California, as well as many piers extending out into the Pacific Ocean. Many of these pile foundations have been designed to carry a safe load of 40 tons per pile.

The development of the oil fields on the shores of southern California required the erection of many piers, some of which extend 1,200 ft. or more out into the ocean. Steel H-piles were used for these piers, and for the supports of the oil derricks. Approximately 500,000 lin. ft. of steel piles have been used for this development during the past five years.

The durability of steel for piles in salt water may be best illustrated by examples of installations that have successfully withstood the action of the salt for many years. Many wharves have been built in Monterey Bay, Calif., erected on steel rails, and used as piles, and these have stood in salt water exposed to rise and fall of the tide for over twenty years. About five years ago a pier was erected out from the shore of the coast of California at Oceanside on steel H-piles, and a recent examination of the piles indicates they will be serviceable for many years to come.

Milo S. Farwell
Pacific Coast Steel Corporation

San Francisco, Calif.
September 6, 1932

SOCIETY AFFAIRS

Official and Semi-Official

Engineers Have Unusual Opportunity and Obligation

In Establishing Self-Liquidating Character of Public Works Before Reconstruction Finance Corporation

To the civil engineers of the country, the Emergency Relief and Construction Act of 1932 affords an opportunity that should not be overlooked. The bill is based on the theory that a part of the wealth of the country now in the form of money should be released for the development of worthwhile community facilities of a productive nature. It provides for the loan of credit by the Federal Government to states, counties, municipalities, and other political subdivisions for the financing of self-liquidating projects, the initiation and construction of which will give immediate employment to local labor and will institute a chain of manufacturing processes. Each of these in turn will give employment wherever machines and materials are manufactured, ore is mined, or transportation is called for.

The Reconstruction Finance Corporation is the Federal agency through which such relief is to be extended, and it is the duty and opportunity of engineers familiar with the field of public facilities to discover such worthy projects and to present them in an appropriate and convincing way to the corporation.

The act under which these works may go forward, definitely requires that they shall be self-liquidating, and that moneys to be loaned on them shall be adequately secured. It is futile to suppose that the Directors of the Reconstruction Finance Corporation, acting on the advice of its Legal Department and its Engineers' Advisory Board, can loan any part of the large sum of $1,500,000,000 in any other way than in accordance with the terms of that act. To present a case to them, therefore, there must be something more than the assertion that the loan is adequately secured, or that the project will be self-liquidating. There must be reasonable demonstration of these facts, and it is in accordance with this idea that the Reconstruction Finance Corporation has issued its circular No. 3, indicating the points to be taken into consideration in determining the self-liquidating character of a project.

Civil engineers throughout the country should therefore view each project from a practical standpoint, and in making applications should submit an engineer's estimate of construction cost, an engineer's estimate of operating and maintenance charges, and an engineer's estimate of earning expectancy. These should accompany the original application. It is illogical to assume that any of the departments of the Reconstruction Finance Corporation will be swayed by persuasive speeches, no matter how good the oratory or how cogent the arguments advanced, unless there is supporting evidence that the project conforms, within reasonable limits, to the provisions of the enabling act. Civil engineers might well consider not only the opportunity afforded them of preparing the basic supporting data, but also the possibility of their having a relation to the project when and if it receives the approval of the Reconstruction Finance Corporation and the necessary funds are forthcoming, either from that source or locally.

The Reconstruction Finance Corporation has announced that it is willing and anxious to make small loans to small communities. A great opportunity is thus presented for engineers to assist these small communities in the preparation of applications. It is our duty as a profession to aid in getting people back to work.

John P. Hogan, *Chairman*
Committee on Public Works

September 20, 1932

Forums for Juniors Arouse Interest

By David M. Hughes

Junior, American Society of Civil Engineers
Executive Secretary, Junior Forum, Los Angeles Section

Members in all grades of the Society have often felt a need of providing something in regular Local Section programs that would be devoted to Juniors and students. The question has always been, what to provide? As a means of affording Juniors and Student Chapter members this greater opportunity to participate in Section activities, Junior Forums are being organized. Although the suggestion, "to be seen and not heard," cannot apply now to Juniors in Los Angeles, they are reminded of it when affiliated with Sections that have not provided places on programs for them.

Organized by Junior Members

Realizing that they might be something more than mere listeners at the usual meetings, Juniors of the Los Angeles Section met for organization on March 12, 1930, inspired by H. W. Dennis, M. Am. Soc. C.E., former president of the Section, and A. F. Barnard, Assoc. M. Am. Soc. C.E., then president of the Section. Ways and means of carrying on a Junior Forum were drafted by a committee of three Juniors, and the next monthly meeting found the Junior Forum a realization, in action under the administration of a rotating committee of three members. It appeared to those entrusted with the method of organization that this would be the most democratic, affording ample opportunity of service to many and being devoid of clique control, as it has since proved to be.

The Forum meets during the hour just prior to the regular Section meeting and provides practically equal opportunities to Juniors and students who are members of the Section; this feature combined with the Forum's Committee on Student Chapters, welds a strong link in the chain of development and interest in progress within the Society organization and the profession.

The aims and objects of the Forum are very broad. It affords Juniors and students opportunities for close association and intercourse in knowledge and opinions on matters of engineering and general interest and provides the means of aiding in the development and welfare of local Student Chapters, and of otherwise furthering the aims of the Section and the Society. Among the means provided are: participation in Junior Forum activities, more particularly in the presentation of papers and other programs and in the conduct of meetings; and assistance to a larger degree in the work of committees of the Section and of the Society. There is no additional financial burden on Juniors and students, as there are no dues.

Interesting programs often attract as many corporate members of the Society as members of the Forum. The variety of subjects discussed is limited only by the future possibilities of ambitious young engineers. Motion pictures of a semi-technical nature are often included. The meetings unquestionably provide an added stimulus to high attendance records for the Section itself and assist materially in the development of the Student Chapters.

Programs Managed by Rotating Board

The plan of administration, although an original development by Juniors, resulting from observation and experience in their own meetings, parallels that of the Society. The administration of the Forum is of necessity more compact; members hold office for short terms so that more may gain experience, and preference of selection is given to members who have not served.

The executive officer of the Forum is the Executive Secretary. His duties are those of president, secretary, and treasurer. The term of office is limited to six months and he cannot succeed himself. A Junior Forum Advisor, usually a former president of the Section, is appointed by the Section president each year. He is the official counselor and representative of the Forum and Section in matters of joint interest. His duties are to act in an advisory capacity, guiding the efforts of the Forum along lines approved by the Society.

A Rotating Board manages the meetings and provides the programs. It is composed of three members: Senior, Intermediate, and Junior. The terms of office are three consecutive meetings, successive and not coincident. One member is appointed as Junior Member at each meeting. He serves as secretary and at the close of his first meeting transmits a résumé, including the roll, to the Executive Secretary. The Junior becomes Intermediate Member at the following meeting, and as such performs the duties of chairman. The next succeeding meeting finds him Senior Member and he then presents a paper or secures some other program. The Senior Member retires at the end of each meeting, and a newly appointed Junior steps into the official line.

Administrative Functions Vested in Board of Control

The administrative body is the Board of Control. It is composed of the Rotating Board, Executive Secretary, last resident Past-Executive Secretary, and Junior Forum Advisor. The Board of Control approves all appointments and disbursements of the Executive Secretary and performs all administrative functions not otherwise delegated. By this arrangement this board has the benefit of past experience and is sufficiently well tempered to prevent any serious missteps. It has also an up-to-the-minute representation, because of the short term of the members of the Rotating Board.

As time passed, more Juniors became interested and demonstrated their potential usefulness for standing and special committees of the Local Section and of the Society. More Student Chapter men attended the Forum and the Section meetings that followed. Minutes and records of the Forum grew larger. A former president of the Local Section was appointed Junior Forum Advisor to begin the year 1932.

New Constitution Adopte

To further stimulate and direct interest, to facilitate appointment of special committees, to efficiently handle information and receive inquiries, necessity dictated another improvement in administration: Having watched the progress of the Forum from the time of its first organization in Los Angeles $2^1/_2$ years ago and having a clear understanding of the requirements for efficient and effective administration, a committee of Juniors, members of the Forum, drafted and presented a new constitution and by-laws, which were approved and adopted by the Los Angeles Section on April 13, 1932. The result is a new Junior Forum, operating successfully under its own constitution and by-laws.

[*For the information or guidance of any Section interested in the formation of a Junior Forum, the Headquarters of the Society will be glad to furnish a copy of the constitution and by-laws of the Junior Forum of the Los Angeles Section. The San Francisco Section has recently inaugurated a similar organization of Juniors and Student Chapter members.*]

Successful Fall Meeting Assured

Plans Mature for Gathering in Atlantic City

Within sound of the pounding surf of the Eastern seaboard, the Fall Meeting of the Society will open at Atlantic City, N.J., soon after this issue leaves the press. By the time many readers open these pages it will be in full swing. Already indications are that there will be a large attendance from every part of the country at the four-day session, beginning October 5, 1932.

The Local Committee has "done itself proud" in providing what promises to be a thoroughly enjoyable round of entertainment to accompany the technical part of the program. Between sessions devoted to the serious business of planning cities and providing for public works construction, those in attendance will have time to enjoy dinner parties, boat trips, golf contests, fishing expeditions, and sight-seeing tours. The committee's plans include elaborate provisions for the entertainment of the ladies and guests at the headquarters hotel and at a nearby country club.

Those who for one reason or another find it impracticable to attend the meeting will await with interest the December issue of Civil Engineering, in which will appear the papers presented there, both those read at the general session and those given before the seven Technical Divisions that are meeting at this time. The articles will be printed as fully as possible in this enlarged issue of Civil Engineering. In this way, members who cannot attend the Fall Meeting will be enabled to read of the latest developments in the profession and in their particular specialties; and those who are able to attend will have a permanent printed record of the Meeting.

Under Auspices of the Philadelphia Section

Six years ago the Philadelphia Section was host to a Society Meeting, when in 1926 the Annual Convention was held in Philadelphia. At that time the Section gave ample proof of the ability of its members as hosts to their fellow engineers. Those planning to attend the coming Fall Meeting are therefore looking forward to a most pleasant as well as a most profitable stay at Atlantic City under the efficient auspices of the Philadelphia Section.

A Word to Junior Members

A Message from the Society's Committee on Juniors

In becoming affiliated with the American Society of Civil Engineers, you have identified yourself with a professional society that has been in active existence for eighty years. Its members have been prominently identified with the development of the United States throughout that period of time, and the influence of the members and of the Society has fostered a continuous growth toward better engineering practice and higher professional standards.

Probably no other engineering society has so jealously guarded its membership qualifications or, as a body, so insistently protected itself against biased or unguarded declarations of opinion or actions.

Welcome to Meetings Extended

As a Junior Member you are welcomed at Society Meetings and given an opportunity to make contacts with older members, men of broad experience, who are often gifted with the faculty of delightful narrative and reminiscence. As a Junior, you are probably somewhat diffident about introducing yourself to older members and hesitant about joining the groups always to be found in friendly conversation at the Society Meetings.

Let us say to you, at this point, that there is also a feeling of embarrassment on the part of the older members in trying to make the acquaintance of the Juniors, a wondering if the young engineer would be interested in those things dear to the heart of the older members of the Society. Very often, the men of long experience, far past their Junior years, crave contact with the younger men of the profession. They know that you have ideas well worth while, new lines of approach to old questions. They are glad of an opportunity for an exchange of ideas.

You will receive from the Society in direct proportion to what you give. Attendance at Society Meetings is a recognized form of investment, as definite as attendance at college, subscriptions to professional magazines, or the building of a professional library.

We send you this greeting, asking that each Junior endeavor to attend Society Meetings and to meet the older members at least half-way in getting acquainted. Call on the members in your vicinity and develop a speaking acquaintance with them. This will soon grow into a delightful friendship, and a better understanding of each other's problems. Engineers need have no fear of unprofessional conduct toward each other, if they develop the possibilities of friendship among the men in their profession.

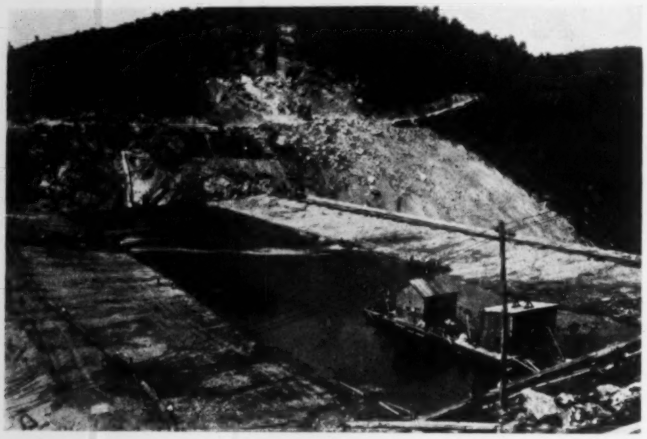
Committee on Juniors
E. N. Noyes, *Chairman*
A. M. Lund J. T. L. McNew
D. A. MacCrea H. P. Treadway

A Preview of Proceedings

The October issue of PROCEEDINGS *will bring to readers, papers on two very different phases of dam construction: the first, a record of the comprehensive tests made to control the clay-puddle core of a large hydraulic-fill dam in New England; the second, a description of the solution of a unique foundation problem for a multiple-arch dam in Mexico. A third paper deals with the preliminary phases of construction of the George Washington Bridge. This will be the second article to appear in* PROCEEDINGS *describing this great structure.*

TESTS FOR HYDRAULIC-FILL DAMS

EXPERIENCE gained by the engineers of the Cobble Mountain hydraulic-fill dam forms the basis of the paper by Harry H. Hatch, M. Am. Soc. C.E., who was in charge of the construction of the reservoir and the laboratory investigations. The paper was presented by Mr. Hatch before the joint session of the Irrigation and Power Divisions on July 7, 1932, at the Yellowstone Convention of the Society. The dam, which was constructed by the City of Springfield, Mass., is 700 ft. long at the crest. The other principal dimensions are as shown in Fig. 1.



SLUICING OPERATIONS ON THE CORE OF THE COBBLE MOUNTAIN DAM

During construction, tests on the filling material, especially the core, were made continuously for checking purposes. The paper describes the methods used in determining the various factors involved. For example, studies to determine the coefficient of friction resulted in the development of a formula in which the coefficient is shown to vary with changes in pressure on the material. Furthermore, percentage of void, consolidation of core material, effective size of materials, seepage, and other characteristics are treated in this paper in a comprehensive and thorough manner. Mr. Hatch states that his principal purpose is to arouse enough interest among engineers connected with hydraulic-fill construction so that methods for testing materials in hydraulic fill dams may ultimately become standardized.

The original manuscript contains 166 typed pages, including 41 illustrations and curves for the ready solution of some of the more complicated equations given in the paper. Limitations of space have prevented the use of the entire manuscript, but the paper as published will be found to be comprehensive in scope, containing a wealth of discussable material. The original document has been filed at the Engineering Societies Library in New York, where it is available for further reference and study.

GEORGE WASHINGTON BRIDGE: ORGANIZATION, CONSTRUCTION PROCEDURE, AND CONTRACT PROVISIONS

IN THE SAME number of PROCEEDINGS, the second paper in the series on the design and construction of the George Washington Bridge is presented. The introductory paper by the Chief Engineer, O. H. Ammann, M. Am. Soc. C.E., appeared in the August number, and it is the plan of the Port of New York Authority to give to the profession its detailed records of this project in a number of papers written in each case by the man who was closest to the particular phase of the work described.

Part of the present paper, which is by Edward W. Stearns, M. Am. Soc. C.E., deals extensively with the subject of traffic surveys: the manner in which they were organized, their scope, the content of the reports, and the conclusions drawn from them. Under "Organization Problems," Mr. Stearns describes the personnel, duties, and problems facing the five engineering divisions. Engineers in general, and especially the members of the Construction Division of the Society, should find much of value in this part of the paper.

Under "Program of Construction" Mr. Stearns discusses several peculiar problems, and in describing the procedure followed, he explains the reasons for it in each case. The paper includes a tremendously important section on the writing of engineering contracts. For example, bonuses were not provided for early completion. On the contrary, penalties were stipulated for failure to follow the prepared construction schedule. The reason for this is that in a work of such magnitude no special benefit would accrue from the early completion of one contract unless all the contracts were likewise completed early.

Proposals were received from five bidders on October 3, 1927. The bids accepted at that time resulted in deciding upon one of the alternate methods of handling the steelwork, as follows: wire cables were adopted in preference to eye-bar cables; second, the cables were to be placed in pairs, side by side, instead of one above the other; and the steelwork was let to two different contractors, one for the cables and the suspended and anchored steel, and the other for the towers and floor steel. Engineers who plan to follow the succeeding papers in this series on the George Washington Bridge should not fail to study Mr. Stearns' paper with particular care.

FOUNDATION TREATMENT AT RODRIGUEZ DAM

A GORGE varying in width from 100 to 750 ft., with side walls of a rock suitable for a concrete dam; a stream bed of a material considered inferior for the foundation of such a structure; and a fault line that follows the stream—these are the conditions that confronted the engineers called upon to design the Rodriguez Dam, at the only available site on the Tia Juana River in Lower California, Mexico.

As explained by Charles P. Williams, M. Am. Soc. C.E., in his paper, the foundation designed for this dam is unique in that it

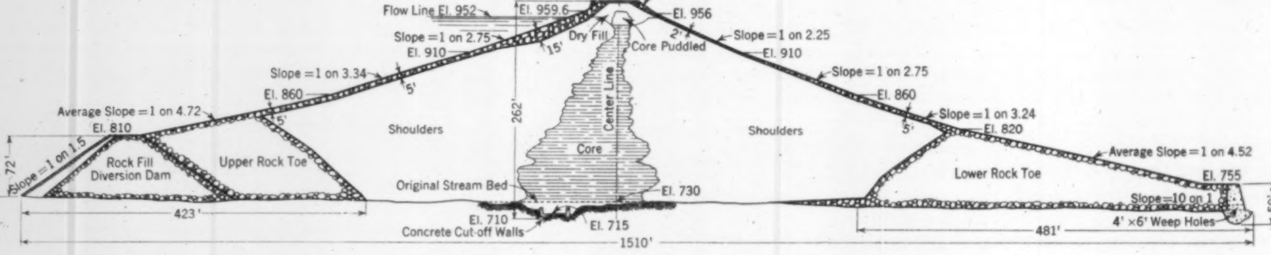
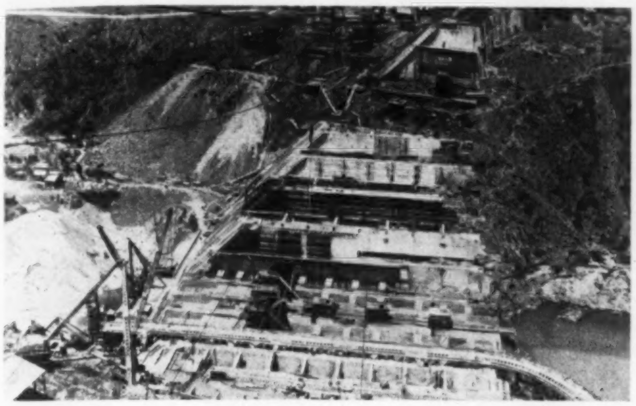


FIG. 1. MAXIMUM CROSS SECTION OF COBBLE MOUNTAIN DAM
Built Across the Little River for the Water Supply of Springfield, Mass.

consists essentially of a wide subterranean arch bridge of reinforced concrete, with abutments planted on the rock at each side of the gorge, so that it spans the less desirable material in the center. Since the space beneath the theoretical arch is filled with concrete, the intrados of the arch is a theoretical line intersecting the canyon walls above the lower limit of the sound rock.

On this man-made foundation, the Ambursen Dam Company is building the dam proper, a slab-and-buttress type, greater in height than any similar structure previously constructed. Mr. Williams confines his remarks to the foundation aspects of the problem, treating the design of the superstructure only sufficiently to show clearly its influence on the foundation.



Rodriguez Dam Under Construction
Tia Juana River, Lower California, Mexico

The drainage basin of the Tia Juana River lies partly in Mexico and partly in San Diego County, California. The river has a drainage area of 1,670 square miles, of which 938 square miles are above the Rodriguez Dam site. It is planned later to build another dam about 40 miles upstream from the Rodriguez site, and this will embrace a drainage area of 554 square miles. Rodriguez Reservoir therefore will be supplied by the run-off from 384 square miles, supplemented by surplus water from above the Cancio Reservoir, and by return flow from about 10,000 acres of irrigated land.

Appointments of Society Representatives

Morrough P. O'Brien, Assoc. M. Am. Soc. C.E., has accepted an appointment to serve on the Society's Special Committee on Irrigation Hydraulics, to fill the vacancy caused by the resignation of Julian Hinds, M. Am. Soc. C.E.

Student Lectures for the Coming Year

On September 15 the letter reproduced below was sent out to the secretaries of the 102 Student Chapters in colleges all over the country. The demand for these lectures in the three school years during which they have been offered, and their growing popularity, has amply justified the continuance of this service for another season.

September 15, 1932

To the Secretaries of the Student Chapters
American Society of Civil Engineers

Dear Sirs:

At the beginning of another school year, I should like to remind you of the material available to all Student Chapters in the form of lantern lectures, which has been found of value in the past three seasons.

The use of these lectures, illustrating various types of engineering construction, has increased from year to year, and during the two semesters of 1931–1932 they were presented at a total of 230 meetings.

There are now ready to send out 13 different titles, as listed below:

Carquinez Strait Bridge
Cascade Tunnel
Catskill Water Supply of New York City
Conowingo Hydro-Electric Development
Coolidge Dam
Florianopolis Bridge
Hetch Hetchy Water Supply and Power Plant
Holland Tunnel
Flood Control in the Miami Conservancy District
Mississippi River Flood Control
Muscle Shoals Hydro-Electric Development
Recent Power Development at Niagara Falls
Westchester County Park System

Reservations will be received at any time, for any date in the school year. It is advisable to make reservations well ahead of the meeting date, as experience has shown that those who wait until the last minute have often been unable to obtain the particular lecture asked for. Postage is paid both ways. A mimeographed lecture accompanies each set of slides, and the material will be forwarded several days in advance, so that the person who is to deliver the lecture may become familiar with the topic, and expand the descriptions, if he so desires.

Yours very truly,
George T. Seabury
Secretary

News of Local Sections

Kansas State Section

At the April 26 meeting of this Section, held in the roof garden of the Kansas Hotel, Topeka, the guests of honor were I. L. Coffman, of the University of Kansas, and F. C. Perry, of Kansas State College, recipients of the 1932 prize award to the honor student of the senior class in civil engineering. A well attended business meeting was held by the Section on August 17.

San Francisco Section

The 163d regular meeting of the San Francisco Section was held at the Engineers' Club on Tuesday evening, June 21. There were 90 members and guests in attendance. A humorous talk about civil engineers was broadcasted by A. Appleton, architect of the firm of Heiman and Appleton. Following a business session, M. M. O'Shaughnessy, consulting engineer, spoke on "General Review of the Hetch Hetchy Project," and L. W. Stocker, Chief Civil Engineer of the Hetch Hetchy Water Supply, spoke on "Recent Construction on the Hetch Hetchy Project."

Spokane Section

The regular monthly noon meeting of the Spokane Section was held on Friday, August 12, at the Davenport Hotel. A very interesting talk on the 40th annual meeting of the Society for the promotion of Engineering Education, held at Oregon State College, was given by Ivan C. Crawford, Dean of the School of Engineering, University of Idaho.

Student Chapter News

Robert H. McNeilly

A series of interesting meetings has been enjoyed by the Robert H. McNeilly Chapter of Vanderbilt University during the recent year. Among the speakers were: F. J. Bauman, of the Tennessee Department of Highways; A. W. Hutchison, a member of the faculty; Maj. Frank Besson, Government Engineer in charge of river control in Tennessee; and Mr. Farrar, president of the Tennessee Road Builders' Association. Society slide lectures shown on various occasions included: "The Development of the New York Water Supply," "Muscle Shoals Hydro-Electric Development," and "The Design and Construction of the Holland Tunnel." The membership of the Chapter is 35.

UNIVERSITY OF CINCINNATI

A report from the Student Chapter of the University of Cincinnati indicates that it has had a successful year. Joint meetings were held with the Cincinnati Section on December 18 and April 28. The subject discussed at the first joint meeting was "The Essential Qualifications of a City Manager," and at the second, "Mississippi River Control." Several meetings were held for consideration of the work on the Cincinnati Union Terminal. The topics discussed at these sessions were "The Terminal Project as a Whole," and "Structural Design Problems on the Cincinnati Terminal Work." Two excursions were taken to the Cincinnati Terminal property under the direction of J. M. Belknap, Field Engineer.

UNIVERSITY OF OKLAHOMA

At a dinner meeting on December 4, at which this Chapter was host to the Oklahoma Section, an address was given by A. R. Losh on "The Engineer and Public Service." A lecture was given by Prof. N. E. Wolford, at the meeting held on March 31, and slides were shown on the "Hetch Hetchy Water Supply and Power Development." On April 28, those present enjoyed a lecture and slides on the "Muscle Shoals Hydro-Electric Development." The lecture was given by Prof. J. F. Brookes.

American Engineering Council

National representative of 27 engineering societies, with a constituent membership of 62,000 professional engineers, reports civil engineering news of the Federal Government

PROGRESS IN STIMULATION OF CONSTRUCTION ACTIVITIES

UNDER DATE of August 27 the Engineers' Advisory Board of the Reconstruction Finance Corporation issued Circular No. 3 of the Reconstruction Finance Corporation, giving information for prospective applicants for loans for self-liquidating projects. This circular gives a detailed interpretation of Title II of the act and should be carefully studied by all persons contemplating making applications for loans. Copies of the circular may be obtained by addressing the Engineers' Advisory Board in Washington. It will be most helpful to the Engineers' Advisory Board and Board of Directors of the Reconstruction Finance Corporation if applications are made in strict accordance with the instructions contained in this circular. The Advisory Board has requested Council to discourage people from coming to Washington to see either members of the Advisory Board or Board of Directors. It is not necessary to come to Washington to make application for loans.

Title III of the Emergency Relief and Construction Act made provision for the expenditure of more than $300,000,000 on public works by the Federal Government. The act provides that before $200,000,000 of this sum can be spent by the various departments, the Secretary of the Treasury must certify that the necessary funds are on hand or reasonably obtainable. This section of the act is of particular interest to all engineers, as resulting appropriations will in large part be used for their employment. [It was reported that Secretary Mills released $200,000,000 on September 9, 1932.]

HOME LOAN BANK BOARD ACTIVE

The board appointed under authority of the recently enacted Federal Home Loan Bank Act has been formulating a definite policy and program under the leadership of F. W. Fort, Chairman. By direction of the act, this board has divided the country into 12 regional bank districts and has fixed the minimum capital of each regional bank so that the new Federal financial unit for the relief of home owners will start operation with a capital of $134,000,000. It is now contemplated that these 12 banks will be in operation and doing business by October 15, 1932.

NATIONAL CONFERENCE OF BUSINESS AND INDUSTRIAL COMMITTEES

At the invitation of President Hoover, leading industrialists and bankers met in Washington on August 26 to consider measures looking toward an increase of business activity, restoration of a higher level of industry and trade, lessened unemployment, and better conditions for agriculture, labor, and the public generally.

The conference adopted a resolution providing for a series of six definite activities as follows:

1. The problem of making available credit affirmatively useful to business.
2. Increased employment on railroads and stimulation of industry through expansion of maintenance of equipment and purchase of new equipment in cooperation with the Interstate Commerce Commission and the Reconstruction Finance Corporation.
3. Expansion of capital expenditures by industry in the way of replacement of obsolete and worn-out equipment, and otherwise.
4. Increased employment through sharing work movement.
5. Possibility of stimulating repair and improvement of home movement.
6. Organization of committees in the several districts to assist home owners with maturing mortgages.

CONFERENCE TO PLAN FOR RELIEF FOR COMING WINTER

President Hoover called a general conference in Washington on September 15, to plan for unemployment relief through the coming winter. The chairman is Newton D. Baker, of Cleveland. The work of this conference is in continuation of the relief activities instituted by President Hoover, such as the drought relief in 1930 and the President's Organization on Unemployment Relief, of which Walter S. Gifford, President of the American Telephone and Telegraph Company, was chairman.

Three subjects of vital interest were discussed at the conference:

1. Ways and means in which states and communities can cooperate in the distribution of the $300,000,000 Federal appropriation for relief.
2. Importance of communities maintaining well-balanced social service programs in the present crisis.
3. Special means to strengthen local fund-raising campaigns, community chests, and others, which last year raised upward of $150,000,000 for welfare and relief purposes.

CONGRESS INVESTIGATING GOVERNMENT COMPETITION

Under authority of House Resolution 235, passed at the recent session of Congress, a congressional committee headed by Representative Shannon, of Kansas City, Mo., is investigating the subject of Government competition with private enterprise. During the summer the individual members of the committee have been making investigations in their own localities. It is expected that formal hearings will be resumed in Washington in September and October.

For some time Council has had a committee studying the effects of Government competition with the private engineer. Council hopes to bring before the Shannon Committee the results obtained by the investigations of its committee. It is to be hoped that engineers in general will bring to the attention of the congressional committee or to Council any information that would help to clarify this issue.

BOARD OF SURVEYS AND MAPS APPROVES RESOLUTIONS OF THE AMERICAN SOCIETY OF CIVIL ENGINEERS

The Board of Surveys and Maps held a special executive meeting in Washington, July 26, to consider and act upon resolutions of the AMERICAN SOCIETY OF CIVIL ENGINEERS, which had been referred to it by the Secretary of Commerce. These resolutions recommended that the President of the United States direct that the program of the U.S. Coast and Geodetic Survey be such as to establish and make available at the earliest practicable date controlling triangulation with the arcs spaced at intervals averaging not more than 25 miles throughout the United States.

The present program of the Survey contemplates the establishment of controlling survey positions resulting from arcs of first-order triangulation spaced at intervals of approximately 100 miles throughout the United States, and of similar positions resulting from arcs of second-order triangulation located approximately halfway between these. The Board's Committee on Control recommended "a policy of spacing the arcs of the national horizontal control net at intervals of approximately 25 miles," substantially in accord with the Society's recommendation.

ITEMS OF INTEREST

Engineering Events in Brief

Civil Engineering for November

THIS YEAR marks an anniversary of the Society, and its beginning on November 5, 1852, will be commemorated in the November issue. On that day eighty years ago, a handful of men met in the old Croton Aqueduct office in New York, adopted a constitution, and named the newly formed organization the "American Society of Civil Engineers and Architects." Thus the Society as we know it today had its beginning. A record of eighty years of progress will be found in the article by the Secretary of the Society, George T. Seabury, M. Am. Soc. C.E., to appear in the November issue.

Another article will be by C. C. Williams, M. Am. Soc. C.E. He reviews the characteristics of vibrations of small amplitude, describes their effect on foundations, and presents the results of some laboratory experiments on model construction.

In the open-air hydraulic laboratory at Walchensee, Germany, an enlightening series of tests on a 1:55 model have recently been completed. Their purpose was to determine the travel characteristics of the rolling debris or detritus carried along the bed of a river and the effect of various levee locations on the movable bed. The tests were conducted under the direction of Dr. Ing. Hubert Engels, with the collaboration of Hans Kramer, Assoc. M. Am. Soc. C.E., then a Freeman Scholar. Lieutenant Kramer's translation of the report of the test will appear in the November issue.

Another phase of the subject of hydraulic engineering will be presented by A. Streiff, M. Am. Soc. C.E., who has written on the utilization of systematic sequences in stream flow to predict run-off. He believes that current views on the fortuity of annual stream flow need to be revised.

The structural engineer will be interested in the article by A. H. Finlay, Assoc. M. Am. Soc. C.E., who has used celluloid models of an arch to determine the effect of the deck in aiding the arch to take the superimposed loads.

Society Cooperates with Engineering Foundation

THE ENGINEERING FOUNDATION is now entering upon its seventeenth year of service, "for the furtherance of research in science and in engineering," and "for the advancement . . . of the profession of engineering and the good of mankind." Established in 1915 by a gift of $200,000 from Ambrose Swasey, Hon. M. Am. Soc. C.E., the Engineering Foundation now has endowments totaling nearly $900,000. The usable income from this is now approximately $40,000 a year.

In its researches, the Engineering Foundation not only enjoys the cooperation of the Four Founder Societies, but also of the National Research Council, the technical bureaus of the Government, of universities, scientific associations, industries, bankers, and individuals. Other funds contributed to projects in which it has participated aggregate hundreds of thousands of dollars, and the services, facilities, and materials given in addition probably could not be purchased for a much larger total.

In the field of civil engineering, the arch dam investigation undertaken in cooperation with the Society, has included tests on full-sized and model dams, and has stimulated international interest and extensive research by others. It is expected that this work, which was begun in 1922, will be completed by the end of this year. Growing out of the arch dam tests, long-time experiments on the plastic flow of concrete are being conducted at the University of California. These tests will be continued for some time.

In connection with concrete arches, cooperative tests of bridges and models, both of single and multiple span, plain and reinforced, have been made in the field and in the laboratory. At the University of Illinois, in cooperation with the Society's Committee on Cement and Reinforced Concrete Arches, particular attention is being given at the present time to multiple arch spans, supported on tall and slender piers. At the University of Wisconsin, in extension of earlier investigations, tests on steel columns for bridges and buildings have been made and analyzed, in cooperation with the Society's Committee on Steel Column Research. In cooperation with the Society's Committee on Earths and Foundations, fundamental and applied studies are being made on all kinds of deposits overlying bedrock. This research is of significance in both design and construction, wherever foundations, embankments, highways, canals, or other types of excavation are undertaken.

In the more general field of engineering education, studies of achievement, aptitude, intelligence, and interest are being conducted in cooperation with the Society for the Promotion of Engineering Education. As a result of this work, the Founder Societies are assisting in the preparation of a pamphlet, "Engineering: a Career a Culture," for aiding boys, parents, and teachers in selecting the proper vocation. This pamphlet is expected to be ready this fall.

The Engineering Foundation has enough work on hand to utilize all its present resources. In addition, the Founder Societies have projects not now being supported that need help from the Foundation. Also, larger appropriations might be used to advantage for some of the work the Foundation is already aiding.

Economics—A Reading List of Books

DURING THE PAST year or two, engineers have shown a marked increase of interest in economics, especially in relation to engineering. Activities and publications of our own Society bear witness to this fact: we have an Engineering-Economics Division; the Society is taking an active part in developing a Normal Program for Public Works Construction to stimulate trade recovery and revive employment; and contributors to these pages have expressed their views on many problems relating to economics.

From the current issue of *Research Narratives*, a pamphlet published monthly by the Engineering Foundation, the joint research organization of the Four Founder Societies, are taken the titles of some recent books on the general subject of economics.

Mention of a book is not intended as an endorsement; omission of one should not be taken to mean disapproval; nor does the order have any significance. The books listed represent a variety of views and may be of some service to readers who wish to pursue the subject.

The Engineer in a Changing Society. An address by William E. Wickenden, President, Case School of Applied Science. *Electrical Engineering*, July 1932, American Institute of Electrical Engineers, New York, $1.

Taming Our Machines. By Ralph E. Flanders. Richard R. Smith, Inc., New York, 1931. $2.50.

Progress and Poverty. By Henry George. The Baker and Taylor Company, New York. New edition, 1931. $1.

Business Versus Finance. By David Cushman Coyle. Published by the Author, New York, 1932. 60 cents.

Business Cycles. By Wesley C. Mitchell. National Bureau of Economic Research, New York, 1927. $6.50.

Health and Wealth. By Louis I. Dublin. Harper Brothers, New York, 1928. $3.

Pathways Back to Prosperity. By Charles Whiting Baker. Funk and Wagnalls Company, New York, 1932. $2.50.

Jobs, Machines, and Capitalism. By Arthur Dahlberg. The Macmillan Company, New York, 1932. $3.

The Swope Plan (for Stabilizing Industry). By Gerard Swope (edited by J. George Frederick). The Business Bourse, New York, 1931. $3.50.

New Roads to Prosperity. By Paul M. Mazur. The Viking Press, New York, 1932. $2.

The World's Economic Dilemma. By Ernest M. Patterson. McGraw-Hill Book Company, New York, 1931. $3.50.

The Tragedies of Progress. By Gina Lombroso. E. P. Dutton and Company, New York, 1931. $3.75.

Toward Civilization. Edited by Charles A. Beard. Longmans, Green and Company, New York, 1930. $3.

Economic Behavior. 2 vols. By Atkins, Edwards, Friedrich. Houghton Mifflin Company, Boston, 1931. $8.50.

Laissez-Faire and Communism. By John Maynard Keynes. New Republic, Inc., New York, 1931. $1.

What Price Progress. By Hugh Farrell. G. P. Putnam's Sons, New York, 1926. $2.50.

Power Control. By H. S. Rausrenbush and Harry W. Laidler. New Republic, Inc., New York, 1928. $1.

Whitewater Canal Aqueduct

IN THE PEACEFUL little town of Metamora, Ind., which was at one time a center of activity, is to be found an old wooden aqueduct that still carries the waters of the "Old Whitewater Canal" $10\frac{1}{2}$ ft. above the stream known locally as Duck Creek. On this canal, built in 1843 and originally connecting Hagerstown and Lawrenceburg, Ind., and later Cincinnati, Ohio, at one time could be seen boats drawn by mules and also mills run by water power. Today, however, all that is left of its activities is a water mill near the aqueduct in Metamora and a paper mill located at Brookville, eight miles away. Near the site of this aqueduct can still be seen two of the old stone masonry locks used in raising and lowering the boats as they traveled through the canal.

Of interest to the civil engineer is the fact that this wooden aqueduct is still carrying its enormous water load across Duck Creek. The aqueduct at present has an average water cross section 17.0 ft. wide and 2.1 ft. deep, with a clear span of 70 ft. This makes an actual water load of 78 tons in normal times. Occasionally this depth is increased to 3.1 ft., making the total water load 115 tons.

The present structure was built in 1847 after the original bridge had been washed away by the high waters of the creek below. On rebuilding, the footings were carried to a greater depth and the masonry for the abutments and wings was set on a grillage of large hand-hewn black-walnut timbers. A few timbers have been added since that time but otherwise very few changes have been made in the structure.

The structure was built originally with a roof for the preservation of the wood timbers, especially at their joints. The siding on the south side was added in 1866, when the canal ceased to carry boats and the railroad laid its tracks along the towpath. The siding was added to prevent the sparks of the wood-burning locomotives from setting fire to the bridge timbers.

Most of the timbers used in this construction are poplar and oak, all hand hewn. The larger timbers are all of poplar. By reference to the sketches in Fig. 1, the size of the timbers as well as the different truss systems may be noted. Most of the timbers are still well preserved except those located where the water continuously drops on them, causing them to rot away. On some of the timbers there can still be seen notches that were worn by the cables dragging against their sides as the mules pulled the boats across the aqueduct. The workmanship on the fitted timbers is very good and shows the skill possessed by the builders of the many wooden structures of that early period.

This information, as well as the accompanying drawings and pictures of this old wooden aqueduct, was furnished through the courtesy of William J. Titus, M. Am. Soc. C.E., Chief Engineer, Indiana State Highway Commission.

STRUCTURE ON BARGE AND POWER CANAL STILL IN USE
Span 70 Ft., Water-Bearing Cross Section 3 by 17 Ft.

WHITEWATER CANAL AQUEDUCT AT METAMORA, IND.
Roofing and Siding for Protection Against Weather

Degrees and Honors

IN ADDITION to those previously published in CIVIL ENGINEERING, the following Society members have recently been awarded engineering honors:

A. GIDEON, M. Am. Soc. C.E., received the Gold Medal of the Metropolitan Water Works of Manila for long and meritorious service. The presentation was made at the recently celebrated fiftieth anniversary of the completion of the city's first water works, the Carriedo System.

JAMES G. STEESE, M. Am. Soc. C.E., was awarded the honorary degree of Doctor of Science by Alaska Agricultural College and School of Mines at its last commencement exercises.

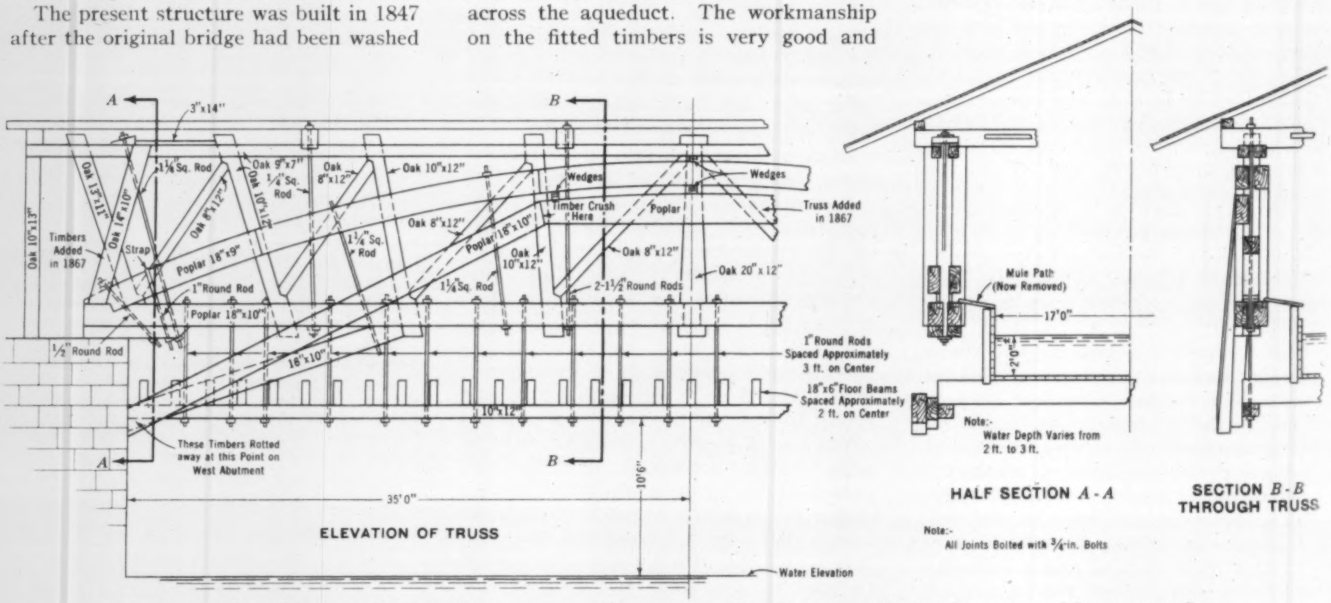

A
3"x14"
B
1⅛ Sq. Rod
Oak 9"x7"
¼" Sq. Rod
Oak 8"x12"
Oak 10"x12"
Oak 13"x11"
Oak 14"x10"
Oak 8"x12"
Oak 10"x12"
Wedges
Wedges
Timber Crush Here
Poplar
Truss Added in 1867
Oak 10"x13"
Timbers Added in 1867
Strap
Poplar 18"x9"
1¼" Sq. Rod
Oak 8"x12"
Poplar 18"x10"
Oak 10"x12"
Oak 8"x12"
Oak 20" x 12"
1" Round Rod
1¼ Sq. Rod
2-1½" Round Rods
Poplar 18"x10"
½" Round Rod
18"x10"
1" Round Rods Spaced Approximately 3 ft. on Center
18"x6" Floor Beams Spaced Approximately 2 ft. on Center
10"x12"
These Timbers Rotted away at this Point on West Abutment
A
B
10'6"
35'0"
ELEVATION OF TRUSS
Water Elevation
Mule Path (Now Removed)
17'0"
2'0"
Note:- Water Depth Varies from 2 ft. to 3 ft.
HALF SECTION A-A
SECTION B-B THROUGH TRUSS
Note:- All Joints Bolted with ¾-in. Bolts

FIG. 1. AN OLD COVERED BRIDGE CARRIES THE WHITEWATER CANAL ACROSS DUCK CREEK, AT METAMORA, IND.

NEWS OF ENGINEERS

From Correspondence and Society Files

O. W. Connet and E. M. Killough have been appointed Consulting Valuation Engineer and Valuation Engineer, respectively, of the Western Maryland Railway Company.

Harry C. Billings has recently been appointed City Engineer and Building Inspector of the City of Claremont, Calif.

W. M. Hall, civil and consulting engineer, has opened offices at 311 Eighth Street, Parkersburg, W.Va.

J. J. Charters, previously employed by Sears, Roebuck and Company, is now connected with the Nassau Suffolk Lumber and Supply Corporation, of Amityville, N.Y.

William F. Smock has been appointed Superintendent of Construction for A. Le Poidevin and Company, Inc., general contractors for the Pioneer Memorial Monument, Harrodsburg, Ky. Formerly Mr. Smock was connected with the engineering department of the Southern Railway Company.

R. F. Hoffmark has accepted a position with the Woods Brothers Corporation of Lincoln, Neb. He was formerly vice-president of A. Guthrie and Company, Inc., Portland, Ore.

F. H. Stephenson is now connected with the Reconstruction Finance Corporation, in Washington, D.C.

Malcolm Elliott, Major, Corps of Engineers, U.S.A., has been relieved from duty at Juneau, Alaska, and assigned to a position in the office of the Chief of Engineers, Washington, D.C.

Melvin Oliver Sylliaasen has been appointed City Engineer of Seattle, Wash. Previously he was Chief Structural Engineer for John Graham, of Seattle.

Thorndike Saville, formerly Professor of Hydraulic and Sanitary Engineering at the University of North Carolina, is now at New York University in charge of this field of study as a new department in the College of Engineering.

A. B. Hargis, who for the past 11 years has been City Engineer of Jellico, Tenn., and a consulting engineer with offices in that city, has resigned his position as City Engineer to become associated with the University of Mississippi as Professor of Civil Engineering and head of the engineering department.

Burton P. Fleming has resigned his position as manager of the Elephant Butte Irrigation District to become Dean of Engineering of New Mexico State College.

C. B. Patterson, formerly connected with A. Bentley and Sons Company of Ohio, is now Assistant Engineer, with the Ohio State Department of Highways.

John Grieve has taken a position as Sales Promotion Manager with the Imperial Varnish and Color Company, Ltd., of Toronto. Formerly Mr. Grieve was General Industrial Sales Manager for Brandram-Henderson, Ltd., of that city.

L. E. Camblos, heretofore an engineer with the A. Raymond Roff Company of Philadelphia, has accepted a position with Herbert H. Conway, Inc., General Contractors and Engineers, Upper Darby, Pa.

F. E. Lamphere has been appointed Engineering Executive to the Engineers' Advisory Board of the Reconstruction Finance Corporation, with headquarters in Washington, D.C. Colonel Lamphere was formerly Construction Manager, United Engineers and Constructors, Inc., of Philadelphia.

George M. Shepard is now Chief Engineer in the Department of Public Works, St. Paul, Minn. Formerly Mr. Shepard was Assistant Chief Engineer for the Metropolitan Drainage Commission of St. Paul.

L. M. Charm, formerly connected with Maurice Blumenthal, Inc., of New York City, has accepted a position with the Stanley Contracting Company, Inc., of Brooklyn.

H. L. Thackwell, consulting engineer of Jacksonville, Tex., has removed his offices to Tyler, Tex., and will continue the practice of engineering from his Tyler headquarters.

N. F. Fenn has been elected vice-president of the South Bay Consolidated Water Company, Inc., of Bay Shore, Long Island, N.Y.

T. R. Agg, former Assistant Dean of Engineering at Iowa State College, has been appointed Dean, to succeed Anson Marston who retired on July 1, 1932.

Changes in Membership Grades

Additions, Transfers, Reinstatements, Deaths, and Resignations

From August 10 to September 9, 1932

Additions to Membership

Abell, DeWitt Sterling (Assoc. M. '32), Asst. Engr., State Board of Health, 5 Marston Rd., Charlotte, N.C.

Behley, Rudolph John (Assoc. M. '32), Constr. Engr., Sperry & Treat Co., New Haven (Res., 31 Prospect St., Milford), Conn.

Boyle, Harold Thomas (Assoc. M. '32), Hillhouse, Miss.

Caldwell, David King (M. '32), County Engr., Houston County; Res. Engr., State Highway Dept., Box 151, Tyler, Tex.

Clarke, Timothy John (Assoc. M. '32), Engr. in Chg., Clarke Bros. Constr. Co., Lake Providence, La.

Connor, John Franklin (Jun. '32), Eng. Asst., Board of Transportation, New York (Res., 87 Grant St., Tompkinsville), N.Y.

Cummins, William Franklin, Jr. (Jun. '32), Box 667, Vicksburg, Miss.

Dalrymple, Tate (Jun. '32), Hydrographer, Board of Water Engrs., Box V, Capitol Station, Austin, Tex.

Davila, Fedrerico (Jun. '32), Box 134, Mayaguez, Puerto Rico.

Edgecombe, Arthur Ralph Brooks (Assoc. M. '32), Faculty Club, Univ. of California, Berkeley, Calif.

Esposito, James Vincent (Assoc. M. '32), Engr., Sun Oil Co., Sales Const. Dept., 1608 Walnut St., Philadelphia, Pa.

Fitzpatrick, Edward Bernard (Assoc. M. '32), Asst. Engr., Westchester County Park Comm.; 1061 Grant Ave., New York, N.Y.

Goldberger, Harold William (Jun. '32), Constr. Engr., Cornell Contr. Corporation, New York (Res., 43–47 One hundred and Fifty-Ninth St., Flushing), N.Y.

Hallock, Harry Earnest (Jun. '32), Insp., State Dept. of Highway, Bureau of Tests (Res., 40 Parkwood Ave.), Columbus, Ohio.

Hapgood, Eugene Palmer (M. '32), City Engr. (Res., 714 North Philadelphia St.), Anaheim, Calif.

Hersey, Edwin Spaulding (Jun. '32), Engr., State Highway Dept., 302 North Division St., Carson City, Nev.

Holt, Arthur Winston (Assoc. M. '32), Care, Westchester Constr. Co., 10 Fiske Pl., Mount Vernon, N.Y.

Johnson, Clarence West (Assoc. M. '32), Chf. Engr., Coastal Constr. Co., Orange (Res., 916 Sixth St., Port Arthur), Tex.

Lane, John Sterling (Assoc. M. '32), 782 Twelfth Ave., San Francisco, Calif.

Livingston, John Joseph (Jun. '32), Cedar Hill, Mo.

Marsh, Ralph Eastman (Jun. '32), Asst. Engr., U.S. Geological Survey, Box 1311, Tucson, Ariz.

Moore, Phillips (Assoc. M. '32), Transitman and Asst. Res. Engr., State Highway Dept.; 406 Park Ave., Lagrange, Ga.

Morgan, Benjamin Arthur, Jr. (Jun. '32), Junior Engr., Municipal Div., The Panama Canal, Box 342, Balboa, Canal Zone.

Mosier, Ray Rosevelt (Jun. '32), 312 South Central, Paris, Ill.

Nelson, Arthur Manfred (Assoc. M. '32), 111 Depot St., Dalton, Mass.

Nelson-Skorniakoff, Theodor (Assoc. M. '32), Chf. Engr., Bureau of Reclamation, Union of Socialistic Soviet Republics (Res., Tverskaia Ulitsa, Staro-Pimenovsky pereulok 14, kv. I-a), Moscow, Union of Socialistic Soviet Republics.

Palmer, Wayne Francis (Assoc. M. '32), Gen. Mgr., The Palmer Steel Co., Inc., 1694 Main St., Springfield (Res., 129 Morgan St., Holyoke), Mass.

Pandya, Anant Hiralal (Jun. '32), Mass. Inst. Tech., Cambridge, Mass.

Pray, Cecil (Jun. '32), With State Highway Dept., Box 55, Brownfield, Tex.

Ranlett, Charles Augustus, Jr. (Jun. '32), Chf. Party, Damon & Foster (Res., Sharon Apartments, Chester Pike and High St.), Sharon Hill, Pa.

Raymond, Allan Tedford (Assoc. M. '32), 1254 Commonwealth Ave., Allston, Mass.

Robbins, Harlan Rowe (Assoc. M. '32), Asst. Engr., California Railroad Comm., State Bldg., San Francisco, Calif.

SCHANCK, CHARLES ARMSTRONG (Assoc. M. '32), Junior Hydrographic and Geodetic Engr., U.S. Coast and Geodetic Survey, Washington, D.C.

SCHMIDT, PAUL HENNING (Assoc. M. '32), Insurance and Valuation Engr., Sinclair Consolidated Oil Corporation, New York, N.Y. (Res., 431 Conrad Rd., Englewood N.J.)

SOLAKIAN, ARSHAG GAZAR (Assoc. M. '32), Lecturer Mech. Eng. and Research Associate, Civ. Eng., Columbia Univ., Physics Bldg., Columbia Univ., New York, N.Y.

WAMSLEY DONALD CARPENTER (Assoc. M. '32), 2d Unit, Santa Fe, Bldg., Dallas, Tex.

WISEWELL, FRANCIS HENRY (Assoc. M. '32), Contr., Engr., 178 Meigs St., Rochester, N.Y.

MEMBERSHIP TRANSFERS

CORCORAN, LOUIS PAUL (Jun. '27; Assoc. M. '32), Field Engr., Portland Cement Assoc., 33 West Grand Ave., Chicago, Ill.

DOMAS, JOSEPH JOHN (Jun. '26; Assoc. M. '32), Asst. Hydr. Engr., Hackensack Water Co., Weehawken, N.J. (Res., 2168 Twenty-Fourth St., Long Island City, N.Y.)

HIGHFILL, JAMES BODENHEIMER (Jun. '29; Assoc. M. '32), Res. Engr., State Highway Dept. Ringling, Okla.

HOYT, KENNETH (Jun. '29; Assoc. M. '32), Engr., William V. Kiehnle Co., 3606 Park Ave. (Res., 3871 Sedgwick Ave., Apartment 4 I), New York, N.Y.

LAMB, GILES HARVEY, JR. (Jun. '23; Assoc. M. '32), With State Dept. of Public Works, Div. of Highways, Box 383, La Jolla, Calif.

OWEN, WILLIAM VICTOR (Jun. '26; Assoc. M. '32), Asst. Engr., Chf. Engr's. Office, New York Telephone Co., 176 Morris St., Albany, N.Y.

POWELL, JOHN EDWARD CHARLES (Jun. '26; Assoc. M. '32), In Chg. Design Squad, Gibbs & Hill, Pennsylvania R. R. Station, New York (Res., 7441 Sixty-Second St., Glendale), N.Y.

SCHOEN HARRY (Jun. '27; Assoc. M. '32), Asst. Secy., Eugene Schoen, Inc., 43 West 39th St. (Res., 30 Olive Pl., Forest Hills), N.Y.

REINSTATEMENTS

LANGTHORN, JACOB STINMAN, JR., Assoc. M., reinstated Sept. 6, '32.

RESIGNATIONS

LATIMER, ROBERT CATHCART, M., resigned Sept. 1, '32.

STROTHMAN, EDWARD CHARLES, Jun., resigned Aug. 16, '32.

DEATHS

ALBERT, FREDERICK WILHELM. Elected Assoc. M., Mar. 4, 1914, M., Jan. 14, 1919; died Aug. 23, 1932.

ALLEN HENRY CLAYTON. Elected Assoc. M., July 1, 1891, M., Jan. 3, 1905; died Aug. 5, 1932.

BROWNELL WALTER KEITH. Elected M., Oct. 14, 1929; died Aug. 5, 1932.

FARLEY, JOHN MOYER. Elected Jun., Dec. 5, 1888, Assoc. M., July 1, 1891, M., Sept. 1, 1897; died Aug. 26, 1932.

McCLAIN, CARL ARTHUR. Elected Assoc. M., Sept. 12, 1916, M., Jan. 16, 1928; died Aug. 5, 1932.

POSEY, GEORGE ADDISON. Elected Assoc. M., Feb. 6, 1912, M., Mar. 16, 1925; died July 30, 1932.

REED, TOM MARTIN. Elected M., June 9, 1930; died Aug. 23, 1932.

ROSS ALBERT HENRY. Elected Assoc. M., Oct. 1, 1928; died July 6, 1932.

ROWSE, ALBERT OWEN. Elected M., Mar. 4, 1908; died Aug. 16, 1932.

WISNER, GEORGE MONROE. Elected M., Feb. 4, 1903; died Aug. 26, 1932.

ZOOK, MORRIS ALEXANDER. Elected M., Oct. 4, 1899; died Sept. 4, 1932.

TOTAL MEMBERSHIP AS OF SEPTEMBER 9, 1932

Members	5,819
Associate Members	6,270
Corporate Members	12,089
Honorary Members	17
Juniors	2,832
Affiliates	118
Fellows	5
Total	15,061

Men Available

These items are from information furnished by the Engineering Societies Employment Service with offices in Chicago, New York, and San Francisco. The Service is available to all members of the contributing societies. A complete statement of the procedure, the location of offices, and the fees is to be found on page 97 of the 1932 Year Book of the Society. Unless otherwise noted, replies should be addressed to the key number, Engineering Societies Employment Service, 31 West 39th Street, New York, N.Y.

CONSTRUCTION

GRADUATE CIVIL ENGINEER; Assoc. M. Am. Soc. C.E.; 43; single; Arizona state license. Experienced in railroad, drainage, and highway location and construction. Available on short notice. D-1445.

CIVIL ENGINEER; Assoc. M. Am. Soc. C.E.; technical graduate; 10 years construction experience on oil refinery and steam power plant, cofferdam, open caisson, and pile foundations. Last 7 years resident engineer for oil refinery in eastern part of United States, with experience in minor design. D-1395.

CIVIL ENGINEER; Jun. Am. Soc. C.E.; 28; single; graduate of Lehigh University; specialized in development work, design, construction, and management; expert surveyor; 4 years experience in general construction work of the type indicated above. Desires position in any capacity, assuring steady work if possible. Available immediately. D-604.

DESIGN

STRUCTURAL ENGINEER; Assoc. M. Am. Soc. C.E.; married; graduate of Lehigh University, 1924; 1 year at graduate school of University of Illinois; $7^1/_2$ years broad experience drafting, designing, and estimating departments with fabricators and engineers on bridges and buildings of all types. Desires position as designer, assistant engineer, or sales engineer in steel or reinforced concrete. C-7922.

CIVIL ENGINEER; Assoc. M. Am. Soc. C.E.; 32; experienced in all phases of structural design; desires position; experience covers work on railroad viaduct, railroad station, elevated streets, subways, pile, caisson, footings, reinforced concrete foundations, mill buildings, bins, conveyor galleries, bridges, etc.; 12 years of experience covers work on both steel and concrete. Hard and conscientious worker. C-8644.

CIVIL ENGINEER; Jun. Am. Soc. C.E.; 31; wants work. Graduate C.E.; licensed; 8 years structural design and estimating. Not afraid of work or long hours. Any location or position. C-1654.

HYDRAULIC ENGINEER; Jun. Am. Soc. C.E.; 30; married; graduate of Massachusetts Institute of Technology; 8 years on design and construction of hydro-electric plants, substations, transmission lines, and distribution structures; and considerable report, valuation, design, and estimating work on hydraulic projects. Prefers design position with public utility or consulting firm. Location immaterial. Available at once. C-3025.

ENGINEERING GRADUATE; Jun. Am. Soc. C.E.; 26; married; 5 years experience; chiefly structural drafting, bridges, and buildings, with large fabricating company. Desires permanent position, preferably on bridge design or construction. D-1289.

STRUCTURAL ENGINEER; Assoc. M. Am. Soc. C.E; 30; married; licensed professional engineer; graduate of Rensselaer Polytechnic Institute; 8 years with consulting engineer—2 years in charge; thoroughly experienced in design, detail, and specifications, fixed and movable bridges, especially vertical lift; structural design for buildings; excellent draughtsman; proficient checker of shop drawings. Available immediately. D-506.

CIVIL ENGINEER; Jun. Am. Soc. C.E; 28; university graduate, 1926; 2 years on road construction and maintenance, as designer and chief of party surveying work; 2 years in building construction, frame houses, apartment houses, as planning engineer and superintendent. Experienced in design of reinforced concrete structures, construction, inspection, and as foreman; desires position in civil engineering work. Salary and location immaterial. D-1433.

STRUCTURAL ENGINEER; Jun. Am. Soc. C.E.; 27; single; graduate with degree of C.E., 1929; 3 years experience in building design, construction, maintenance, and estimation. Familiar with road and track work. Desires permanent situation, designing or construction supervision. Location North or East. Available immediately. D-1456.

EXECUTIVE

ENGINEER-EXECUTIVE ON WATER WORKS AND CONSULTING ENGINEER; M. Am. Soc. C.E.; 41; chief engineer for 8 years on large Middle-Western water works; designed and supervised $2,000,000 water-works expansion; obtained outstanding operating efficiency. Wide consulting engineering experience in municipal improvements, including legal procedure. Desires responsible charge. Can produce results. Location New Jersey or near-by states. D-1340.

CIVIL ENGINEER; Jun. Am. Soc. C.E.; 29; married; graduate of Lehigh University; 8 years experience on surveys, design, estimates, and construction of private and municipal engineering projects—street, water systems, storm and sanitary sewers, small sewage treatment plants, waterfront improvements. Capacity for detail. Desires responsible position with contractor or consulting engineer. D-1400.

EXECUTIVE; Assoc. M. Am. Soc. C.E.; licensed structural engineer and construction superintendent; 19 years varied experience in design, estimates, and construction of reinforced concrete, structural steel, and timber buildings, bridges, tipples, head frames, bins, conveying equipment, grain elevators, flour mills, and plant layouts. Exceptional experience on concrete form work. D-1415.

CIVIL AND STRUCTURAL ENGINEER; Assoc. M. Am. Soc. C.E.; married; licensed professional engineer, state of New York; 25 years experience in design and construction of hydro-electric and steam power plants, transmission lines, industrial plants, electric railways, estimates, specifications, and purchasing. Desires responsible charge of work. Location New York. B-5423.

CONSTRUCTION ENGINEER; Assoc. M. Am. Soc. C.E.; 46; married; 29 years experience. Commercial and industrial buildings—layouts, design, construction, purchasing, and contact work in responsible charge for architects or owners. Can handle complete building project and maintenance for owner or corporation, manage office, and contact clients for architect, engineer, or contractor. Permanency desired. D-1436.

Industrial Engineer and Architect; M. Am. Soc. C.E.; graduate; licensed two states. Supervision of office and field forces, and organization of staffs. Design and construction of housing groups, and water and sewer systems and all other utilities in connection therewith; commercial garages, hydro-electric developments, industrial plants, and preliminary investigations and reports. B-2835.

Structural Engineer Executive; M. Am. Soc. C.E.; 45; married; over 20 years successful experience directing estimating, designing, and sales of steel fabricating business in Ohio and western Pennsylvania. Available immediately. C-5095.

Graduate Civil Engineer; Assoc. M. Am. Soc. C.E.; 8 years varied structural experience on drafting, design, and construction; estimates, inspection, surveys, layouts, and related work. Readily adaptable to new work. Desires connection with construction company, consultant, contractor, architect, in field or office; also sales-engineering or instructorship. C-2605.

JUNIOR

Junior Engineer; Jun. Am. Soc. C.E.; 25; married; B.S. in C.E.; 5 years railroad experience, as rodman and transitman on location and construction of new lines, including responsible charge of ten-mile grading and bridging job. Assistant engineer and accountant on valuation work. Location immaterial. References. D-1385.

Civil Engineer; Jun. Am. Soc. C.E.; 22; honor graduate of Virginia Military Institute in 1931; 6 months experience as estimator and inspector on construction work with general contractor; 6 months office work. Desires position in any branch. Excellent references. Location immaterial. Available immediately. D-1399.

Civil Engineer; Jun. Am. Soc. C.E.; 28; single; B.S. in C.E.; 5 years varied railroad experience. Desires position with any engineering firm which recognizes initiative and industry. Location New York City. C-1538.

Graduate Engineer; Jun. Am. Soc. C.E; 29; married; over 4 years railroad experience in maintenance, including field work on drainage, preliminary and topography surveys, and construction; office work, consisting of working-up and plotting field notes, making and handling engineering records, and drafting. Location immaterial. Salary reasonable. Available immediately. D-1449.

Graduate Civil Engineer; Jun. Am. Soc. C.E.; 24; single; graduate of University of Illinois; 17 months experience on buildings under construction as foreman, mechanic. Can read plans for construction and estimating. Desires work with construction company or engineering firm. References on request. Available any time. Location immaterial. D-1469.

Civil Engineer; Jun. Am. Soc. C.E.; 26; single; degrees of B.A., B.S., and C.E. from Columbia University, graduating with highest honors; 1$^1/_2$ years experience in building construction; 6 months surveying; desires position with company doing design, construction, or research work. Available immediately. C-5309.

MISCELLANEOUS

Operating Engineer on Water Supply; Assoc. M. Am. Soc. C.E.; 45; married; member American Water Works Association; registered civil engineer and certified assistant hydraulic engineer, state of California; 20 years experience—appraisals, estimates, organization, general management of construction, and operation of public and industrial water supplies, also irrigation systems. Available immediately. Location preferred, Pacific Coast. D-1411-328-A-8.

TEACHING

Instructor; Assoc. M. Am. Soc. C.E.; 28; would like position as instructor or engineering assistant. Has first and second degrees in civil engineering; 5 years experience in teaching civil engineering and engineering drawing; 3 years as topographical and highway draftsman; 3 months research in sanitary and hydraulic engineering. D-1460.

RECENT BOOKS

New books of interest to Civil Engineers recently donated by the publishers to the Engineering Societies Library, will be found listed here. A comprehensive statement regarding the service which the Library makes available to members is to be found on page 87 of the Year Book for 1932. The statements made regarding the books are taken from the books themselves and this Society is not responsible for them.

Annuaire des Ingénieurs de France, 1931. Paris, Loubat, Éditeur. 1070 pp., 9 × 5 in., paper.

A directory of about 35,000 names and addresses, including the engineers who are graduates of the 12 principal engineering colleges of France, the members of the French Society of Electricians, and of the Society of Civil Engineers of France.

Architectural Acoustics. By V. O. Knudsen. New York, John Wiley & Sons, 1932. 617 pp., illus., diagrs., charts, tables, 9 × 6 in., cloth, $6.50.

A reference work in which the theory and practical applications of architectural acoustics are clearly set forth. Much information is given on the absorption of sound, sound insulation, and properties of insulating materials. The application of acoustics to schools, churches, theaters, and other buildings is discussed at length and illustrated by numerous examples.

Aviation and the Aerodrome. By H. A. Lewis-Dale. Philadelphia, J. B. Lippincott Co., 1932. 168 pp., illus., diagrs., charts, tables, 9 × 6 in., cloth, $6.

This book deals concisely and definitely with the question of sites and the broad engineering problems involved in the construction and ground equipment of aerodromes and includes examples of plans and buildings. The author has been engaged since 1914 in constructing aerodromes for the British Admiralty and Air Ministry.

Das Kräfteplan-Verfahren. By J. D. Gedő; Leipzig, Alfred Kröner Verlag, 1932. 184 pp., diagrs., tables, 10 × 7 in., 12 rm.

This work develops an original process for the calculation of statically indeterminate structures, which reduces to a minimum the work of integration and simplifies the solution of the equations. The process is entirely an analytic one, and its application to rectangular frameworks, continuous beams, Vieredeel girders, and buildings is illustrated. Formulas for these are developed.

Der Grundbau. By A. Schoklitsch. Vienna, J. Springer, 1932. 490 pp., illus., diagrs., charts, tables, 11 × 8 in., cloth, 62 rm.

A reference book for engineers and students which aims to give a comprehensive picture of modern theory and practice in foundation work. The author discusses practically and in detail the properties and behavior of soils, foundation materials, and their behavior in water and earth, sheet piling, excavating, the preparation of the ground for foundations, special types of foundations, and waterproofing. A wealth of drawings and photographs adds much to this valuable review of the art, which is dedicated to John R. Freeman, Hon. M. Am. Soc. C.E.

Die Abhangigheit der Werkstoffdämpfung von der Grösse und Geschwindigkeit der Formanderung. (Mitteilungen des Wöhler-Instituts, Heft 11.) By E. Bankwitz. Berlin, NEM-Verlag 1932. 53 pp., illus., diagrs., charts, tables, 8 × 6 in., paper, 3.60 rm.

New data on the behavior of metals under repeated stress, with vibration diagrams of steel, copper, brass, and aluminum, based on careful tests, and discussion of the influence of speed of deformation on magnitude of damping, and of alternating stresses on the static properties of these materials.

Die Jahresausgaben von Gruppenwasserwerken und Deren Verteilung. (Beihefte zum Gesundheits-Ingenieur, Reihe 2, Heft 12.) By A. Eigenbrodt. München and Berlin, R. Oldenbourg, 1932. 28 pp., diagrs., charts, tables, 12 × 9 in., paper, 4.80 rm.

A discussion of proper methods of distributing annual operating expenses among a group of water works.

Highway Materials. By E. E. Bauer. 2 ed. New York and London, McGraw-Hill Book Co., 1932. 374 pp., illus., diagrs., charts, tables, 9 × 6 in., cloth, $3.50.

This book intended primarily as a college text, aims to give the student a general working knowledge of the qualities of materials desired for various types of pavements. Methods of sampling and testing, qualities and specifications of road materials, and interpretation of results of tests are discussed. The specifications and tests of the American Society for Testing Materials are used, and this edition is revised to include its latest practice.

International Association for Bridge and Structural Engineering. Preliminary Publication First Congress, Paris, 1932, May 19–25. Published by General Secretariat, Swiss Federal Institute of Technology, Zurich, 1932. 683 pp., illus. diagrs., charts, tables, 10 × 7 in., cloth. 25 Swiss frs. for members; 35 Swiss frs. for non-members.

A preliminary report of the first congress of this association, which occurred at Paris last May, containing the papers prepared as a basis for discussion at the congress.

The topics selected for discussion were: stability and strength of structural members under compression; slabs in reinforced-concrete construction; welding structural steel; large reinforced-concrete girder bridges; dynamics of bridges; influence of the physical properties of materials on the statics of reinforced concrete; use of girders with concrete; research on building ground. The papers are in English, French, and German, with summaries in all three languages.

Smoke: A Study of Aerial Disperse Systems. By R. Whytlaw-Gray and H. S. Patterson. London, Edward Arnold & Co., 1932. 192 pp., illus., diagrs., charts, tables, 9 × 6 in., cloth, 14s.

Although much attention has been directed to practical problems connected with smoke, few attempts have been made to explain the behavior of smoke in terms of general principles, or even to correlate its properties with the number, size, and nature of its constituent particles. The present volume is a contribution to the scientific study of smoke, based largely on investigations carried out at Leeds during the last few years. It provides a general survey of the subject, describes the methods of investigation and the results obtained, and points out lines for further research.

Stresses in Simple Structures. By L. C. Urquhart and C. E. O'Rourke. 2 ed. New York and London, McGraw-Hill Book Co., 1932. 339 pp., diagrs., charts, tables, 9 × 6 in. cloth. $3.50.

A textbook dealing with the fundamentals of stress calculation in simple structures, without considering economics of design. Both analytical and graphical methods are used. The new edition has been entirely rewritten, and chapters have been added on the three-hinged arch and on deflection.

Western and Atlantic Railroad of the State of Georgia. Compiled by J. H. Johnston. Atlanta, Ga., Public Service Commission, 1932. 364 pp., illus., charts, tables, maps, 9 × 6 in., leather, $5.

This volume traces the history of a state-owned railroad from its inception to the present day and describes its varying fortunes under state operation and in the hands of successive lessees. The volume is based upon official records and is primarily intended to supply legislators of Georgia with a convenient record of essential facts. It is also an interesting addition to the history of American railroads.

Winden und Krane, Aufbau, Berechnung und Konstruktion. Heft 5. By R. Hänchen. Berlin, J. Springer, 1932. pp. 307–400, illus., diagrs., charts, tables, 11 × 8 in., paper, 8 rm.

The design and construction of gantry, wall, jib, and pillar cranes are discussed in a thoroughly practical manner in this section of Hänchen's handbook. Numerous drawings illustrate a wide variety of designs, and the calculations are elucidated by completely worked examples. A bibliography is included.

CURRENT PERIODICAL LITERATURE

Abstracts of Articles on Civil Engineering Subjects from Magazines in This Country and in Foreign Lands

Selected items from the current Civil Engineering Group of the Engineering Index Service, 29 West 39th Street, New York, N.Y. Every article indexed is on file in The Engineering Societies Library, one of the leading technical libraries of the world. Some 2,000 technical publications from 40 countries in 20 languages are received by the Library and are read, abstracted, and indexed by trained engineers. With the information given in the items which follow, you may obtain the article from your own files, from your local library, or direct from the publisher. Photoprints will be supplied by this library at the cost of reproduction, 25 cents per page, or technical translations of the complete text may be obtained when necessary at cost.

BRIDGES

Arch Construction. Method of Arch Construction, E. Miozzi. *Concrete and Constr. Eng.*, vol. 27, no. 8, Aug. 1932, pp. 461–467. Article similar in contents to one previously indexed from *Annali dei Lavori Pubblici*, Dec. 1931; determination of position of systematic incisions; angle of opening of systematic incisions; application to Druso Bridge; method of construction. Published by Int. Ass'n for Bridge and Structural Eng.

Arch Stresses. Calculation of Stresses in Arches with Fixed Ends, J. D. W. Ball. *Engineer*, vol. 154, no. 3994, July 29, 1932, pp. 104–106. This method, which is partly graphical, possesses the advantage of visual control of errors of any magnitude, but any reasonable degree of accuracy can be obtained by careful drawing to a sufficiently large scale.

Canada. Deck Type Plate-Girder Bridge at Vidal Street, Sarnia, Ont., R. C. Manning. *Can. Engr.*, vol. 63, no. 5, Aug. 2, 1932, pp. 5–6 and 16. Description of plate-girder highway bridge 254 ft. long with a maximum center span 72 ft. long and 40 ft. wide; details of steel-grid floor.

Combined, Denmark. Proposed Bridge Across Storstroem, Denmark. *Engineering*, vol. 134, no. 3471, July 22, 1932, p. 104. Preliminary design for a longer bridge across Storstroem calls for a height of 26 m.; there are three central spans of steel, the middle one being 160 m. and each of the two outer ones 130 m.; the remainder is of reinforced concrete, with spans of from 40 to 80 m. The bridge, with a total length of 3,300 m., will be the longest in Europe; the smaller bridge over Masnedsund will be 200 m. long, with a maximum opening of 25 m.

Concrete Arch, Great Britain. Twickenham Bridge, W. L. Scott. *Concrete and Constr. Eng.*, vol. 27, no. 8, Aug. 1932, pp. 434–445. Design and construction of concrete arch highway bridge, with total length of 1,253 ft., including approach viaducts; maximum arch span 103 ft. 4 in.; width between parapets 70 ft.

Construction. Construction of Chiswick Bridge. *Roads and Road Construction*, vol. 10, no. 114, June 1, 1932, pp. 170 and 171. Construction of highway bridge over Thames River; 607 ft. total length; center span of 150 ft. clear; total width 70 ft.

Great Britain. Curious Old Timber Viaducts, Great Western Railway, England. *Eng. News-Rec.*, vol. 109, no. 5, Aug. 4, 1932, pp. 131 and 132. Description of timber viaducts, about 80 years old, on Great Western Railway, England, which are to be renewed this year; timber struts radiating fan-shape from tall masonry piers carry built-up stringers for 66-ft. spans; complicated ironwork details.

India. Bridge Construction with Particular Reference to Foundations in Indian Conditions, C. I. Stabler. *Indian Eng.*, vol. 92, no. 26, 1, 2, and 3, June 25, 1932, pp. 514–516; July 2, pp. 15–17, July 9, pp. 32–35, and July 16, pp. 52–54, 9 supp. pp. Review of Indian practice in design and construction of bridge cofferdams, pile and well foundations, well curbs; pier caisson; filling of dredge wells; construction of piers, stagings, and falseworks. (Continuation of serial.)

Lift, Great Britain. Tees (Newport) Bridge. *Surveyor*, vol. 82, no. 2112, July 15, 1932, p. 43. Description of Great Britain's first vertical lift bridge, now under construction, which will have a main lift span 270 ft. long, a vertical range of 100 ft., and a carrying roadway 38 ft. wide plus two sidewalks 9 ft. each in width.

Pennsylvania. Building America's Longest Concrete-Arch Bridge. *Eng. News-Rec.*, vol. 109, no. 3, July 21, 1932, pp. 67–69. Survey of modern construction practices as employed on the George Westinghouse Bridge at East Pittsburgh, consisting of five arches, having a total length of 1,560 ft., and including one span 460 ft. center to center of piers, with a rise of 156 ft.

Piers, Wrecking. Blasting Effective in Pier Demolition. *Railway Eng. and Maintenance*, vol. 28, no. 8, Aug. 1932, p. 497. Details of demolition of three reinforced concrete bridge piers of Pennsylvania Railroad, located midstream of Delaware River, about 2 miles below Trenton, N.J.; explosive loading planned carefully; plan and elevation of pivot pier, showing method of loading for blast.

Plate Girder, Continuous. Long Continuous Girders at Cincinnati Terminal, T. P. Young. *Eng. News-Rec.*, vol. 109, no. 4, July 28, 1932, pp. 110–112. Bridge, with 146-ft. girders extending over three spans, forms part of south approach to new union passenger station at Cincinnati; erection of 45-yd. girders over busy street.

Railroad. Repairs to Irvine River Viaduct, L.M.S.R. *Railway Engr.*, vol. 53, no. 631, Aug. 1932, pp. 288–290. Characteristics of reconstruction work to stone arch viaduct on the main line between Glasgow and Ayr, where railroad crosses Irvine River; diagrams illustrating repair work.

Steel, Welding. All-Welded Steel Bridges. *Ry. Gaz.*, vol. 57, no. 1, July 1, 1932, pp. 24–26. First bridges in England to be constructed entirely by welding in place of riveting are on the L.N.E.R.; general design and construction features.

Strengthening and Repairing. Use Arc Welding in Strengthening and Repairing Metal Bridges. *Ry. Eng. and Maintenance*, vol. 28, No. 7, July 1932, pp. 444–447. Erie, Pennsylvania, and Long Island railroads find process to be economical and effective in solving various problems; columns strengthened and repaired.

Structural Steel, Welding. Arc Welding, A. G. Leake. *Conn. Soc. Civ. Engrs. Proc.*, 1931, pp. 71–83. Review of recent practice as exemplified in the following structures; pedestals, boom struts, bridges—all for the New York Central Railroad; Morris Heights Station, New York Central Railroad; Bridges at Ashland, Pavonia, and Caledonia, Ohio, Erie Railroad, etc.

Suspension. Transporter Suspension Bridge to Thrill World's Fair Visitors, W. G. Grove. *Eng. News-Rec.*, vol. 109, no. 6, Aug. 11, 1932, pp. 172 and 173. Description of so-called "skyride," novel cable and rope suspension system, on 1,850-ft. span between towers 600 ft. high, which will carry rope tracks at a 200-ft. level, upon which 32 passenger cars will run.

Trestle. Concrete Superstructure Built on Old Timber Piling, T. D. Hunt. *Eng. News Rec.*, vol. 109, no. 7, Aug. 18, 1932, p. 195. Reconditioning of old trestle on east shore of Lake Washington near Seattle, Wash; reconstruction cost of $20 per lin. ft. made possible by closely spaced bents, permitting simple short-span concrete deck slab.

Viaducts, Concrete. Sixth Street Viaduct, Los Angeles, M. Butler. *West Construction News and Highways Bldr.*, vol. 7, no. 13, July 10, 1932, pp. 385–391. Concrete and steel structure 3,500 ft. long, over the Los Angeles River, constructed at a total cost of $2,500,000; to connect Whittier Blvd. with Sixth St.; spans are as long as 81 ft.; details of piers.

Welding. Notes on Arc Welding in Bridge Work in Australia, W. D. Chapman. *Iron and Coal Trades Rev.*, vol. 124, no. 3355, June 17, 1932, p. 995. Review of the application of arc welding to the repair and construction of railway and highway bridges in Australia; main applications in repair work; new railroad bridges; types of electrodes. Before Int. Ass'n of Bridge and Structural Eng.

BUILDINGS

Earthquake Effects. Lifting and Restoration of Three-Story Reinforced Concrete Building, J. S. McAven. *Structural Engr.* vol. 10 (New Series), no. 8, Aug. 1932, pp. 326–332. Reconstruction of New Zealand building badly damaged by recent earthquake.

Railroad Stations, Reconstruction. Difficult Building Column Changes Made Successfully. Ry. Age, vol. 93, no. 5, July 30, 1932, pp. 147–149. Unusual alteration problem in New York City; 250 to 1000-ton loads transferred to new support to permit track and platform changes; methods employed; plan showing column, platform, and track changes made.

Steel, Electric Welding. Latest Structural Welding Developments, F. P. McKibben. *Can. Engr.*, vol. 63, no. 3, July 19, 1932, pp. 15–17. Welding and building codes; arc welding of Pediatric Building, Boston; qualification tests for welders; preliminary estimates of welding; welding data for multi-story buildings; inspection of welded buildings.

CITY AND REGIONAL PLANNING

England and Australia. Town Planning in England and Australia, J. N. Yeates. *Commonwealth Engr.*, vol. 19, no. 11, June 1, 1932, pp. 383–387. General review of English and Australian practice with examples of recent planning, and a discussion of legislative acts.

Great Britain. Clyde Valley Regional Planning Scheme, F. A. B. Preston. *Inst. Mun. and County Engrs. Journal*, vol. 59, no. 2, July 19, 1932, pp. 93–114 (discussion), pp. 114–121. History of region; project of rational plan for an industrial region which is famous for shipbuilding and coal mining.

New York. Studies in Super-Block Housing. *Arch. Forum*, vol. 57, no. 2, Aug. 1932, pp. 161–166. Two studies of possible rehabilitation of a portion of the Lower East Side of New York City, made at New School for Social Research's "Architectural Workshop"; financial setup; traffic thoroughfares.

Rome, Italy. Il piano Regolatore di Rome, V. Civico. *Ingegnere*, vol. 6, no. 5, May 1932, pp. 307–316. (1 supp. p.) Outline and discussion of a new plan for the city of Rome, described in several previously indexed articles.

Switzerland. La reconstruction de la Rive droite à Genève, M. Braillard. *Bul. Technique de la Suisse Romande*, vol. 58, no. 5, Mar. 5, 1932, pp. 53–58. Features of new plan for right shore district of Geneva, Switzerland, with special reference to architectural and transportation requirements.

CONCRETE

Aggregates, Testing. Concrete Water Control Apparatus, C. L. Brock. *Rock Products*, vol. 35, no. 13, July 2, 1932, pp. 44 and 45. Features of instruments for the determination of moisture and the specific gravity of sand and other concrete aggregates.

Arches. Some Considerations in Design of Hingeless Arches, C. V. Wolff. *Concrete and Constr. Eng.*, vol. 27, no, 7, July 1932, pp. 383–397. Theoretical mathematical discussion; general form of arch; effect of $l_s:l_c$ ratio; effect of rib-shortening term; use of curves; effect of rise to span ratio.

Construction. How to Avoid Faulty Concrete in Small Structures. *Purdue Univ.—Eng. Extension Dept.—Series No. 27*, vol. 16, no. 1, Jan. 1932, 154 pp. Elementary discussion of practical concrete construction covering the following subjects: kinds of concrete; buying and handling cement; harmful substances in Indiana aggregates; water and consistency; estimating quantities of materials; joints; cracks in concrete; cold weather methods; faulty concrete construction; specifications; inspecting and testing concrete; stucco, etc. Bibliography.

Design. Analysis of Bending and Shearing Forces on Circular and Rectangular Reinforced Hollow Concrete Sections, G. J. Gumersall. *Structural Engrs.*, vol. 10 (New Series), no. 8, Aug. 1932, pp. 342–352. Theoretical mathematical discussion.

ONE length of 48-inch cast iron pipe weighs over 6 tons. To produce 16 lengths it is necessary to mine 275 tons of ore, 275 tons of coal and 75 tons of limestone. This material—625 tons—yields 100 tons of pig iron which in turn yields 100 tons of pipe. Freight is ultimately paid on 825 tons of material. To produce and lay 100 tons of cast iron pipe requires the labor of 740 men for a day. Laying pipe thus gives employment in basic industries—mining foundries, railways, contracting—and is one of the ways back to prosperity.

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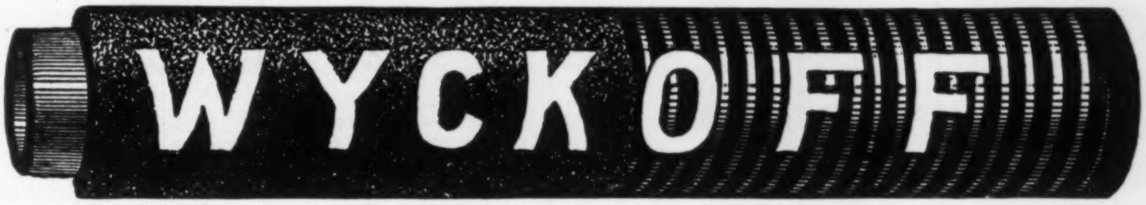

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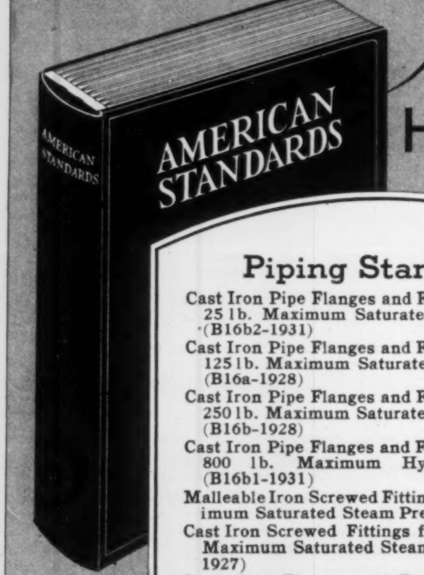

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Hoover Dam Project, Concrete Construction. Concrete for Hoover Dam—II, W. R. Nelson, *Reclamation Era*, vol. 23, no. 7, July 1932, pp. 125–127. Description of concrete mixing plant; control of aggregate quantities; mixing operations; plant equipment.

Ready-Mixed. Ready-Mixed Concrete Finds Place in B. & B. Work. *Ry. Eng. and Maintenance*, vol. 28, no. 7, July 1932, pp. 451 and 452. New York Central utilizes concrete to good advantage on jobs involving paving and building foundation work; ready-mixed concrete speeded building job; use of ready-mixed concrete economical and advantageous.

Silos, Concrete. Reinforced Concrete Storage Bins, J. M. Jardine. *Indian Eng.*, vol. 91, no. 26, June 25, 1932, pp. 512–514. Review of Indian practice in the design and construction of concrete silos and bins for storage of rice, grain, and coal.

Slide Rules. Slide Rule for Reinforced Concrete. *Engineering*, vol. 134, no. 3472, July 29, 1932, p. 120. The firm of A. W. Faber, Stein, near Nuremberg, has prepared a new slide rule specially designed for the needs of the draftsman engaged in the design of reinforced concrete; it is at the same time a general-purpose rule, having the usual *A*, *B*, *C*, and *D* scales.

Testing. Concrete Test Cubes, W. L. Scott. *Concrete and Constr. Eng.*, vol. 27, no. 7, July 1932, pp. 375–379. Methods of sampling concrete and correct preparation of tubes for crushing tests to insure uniform results.

CONSTRUCTION INDUSTRY

Costs. Unit Bid Summary. *West Construction News and Highways Bldr.*, vol. 7, no. 14, July 25, 1932, pp. 430, 432, and 434. Unit costs bid on bridge construction, and street and road work in California, Oregon, Washington, and other Western states.

DAMS

Concrete, Utah. Provo Equalizing and Storage Reservoir, C. J. Ullrich. *Eng. News Rec.*, vol. 109, no. 3, July 21, 1932, pp. 63–65. Construction of covered reinforced concrete reservoir, with capacity of 5 million gal., for storing spring water supply; the reservoir consists of six segments of circle between copper expansion joints of the U-bend and the felt filler; it does not depend on hoop action of reinforcing; all inlets and outlets were brought in under walls; metal-shield expansion joints and mopped waterproofing prevent leaks; total cost $90,000.

Concrete Gravity, Construction. Belt Conveyors Place Concrete for Vermunt Dam in Alps, B. Widman. *Eng. News-Rec.*, vol. 109, no. 5, Aug. 4, 1932, pp. 126–129. Construction of Austrian gravity concrete dam, of 172 ft. maximum height, and volume of 188,000 cu. yd.; using no-slump concrete, placing was accomplished by belt conveyors that delivered steady stream from mixers to form to thin layers that could be tamped without shovel distribution; electric control for backing and mixing unit.

Construction. Construction of Hoover Dam, C. Lake. *Compressed Air Mag.*, vol. 37, no. 6, June 1932, pp. 3834–3839. Use of compressed air in rock drilling, tunnel driving, and concrete construction; use of compressed air for extraction of heat from concrete to accelerate setting; 150 miles of 2-in. piping will be embedded in dam to circulate cooling water through concrete to extract heat accompanying setting.

Earth, California. Motorized Equipment Averages 10,000 Yd. a Day in Building Rolled-Earth Dam. *Construction Methods*, vol. 14, no. 7, July 1932, pp. 30–32. Construction of Santiago Creek Dam in Orange County, California, 130 ft. high above stream-bed, containing over 1,100,000 yd. of material; conveyor 2,600 ft. long, made up of five sections, each equipped with a 30-in. belt driven by a 25-hp. motor at a speed of 375 ft. per min., delivers impervious fill material to carts drawn by 60-hp. tractors.

Foundations. Wire Saws Cut Rock Trench for Dam, C. B. Cornell. *Construction Methods*, vol. 14, no. 7, July 1932, pp. 38 and 39. Use of wire saws in cutting sides of cut-off trench of the Prettyboy Dam for the Department of Public Works of the City of Baltimore; difficulties of channeling; wire-saw operation; calyx drill hole.

Hoover Dam Project, Penstocks. Hoover Dam Penstocks Set Record for Size. *Eng. News-Rec.*, vol. 109, no. 3, July 21, 1932, pp. 79 and 80. Main-headers to serve power units which will be 30 ft. in diameter; arc welding specified for all longitudinal joints and stiffeners; two new 37-ft. tunnels required; construction program and schedule; pipe fabrication and testing.

Pennsylvania. Pymatuning Reservoir Project, C. E. Ryder. *Engr's. Soc. Western Pa.—Proc.*, vol. 48, no. 5, May 1932, pp. 103–140 (discussion) 140–153. Construction of reservoir, 26 sq. miles in surface area in the headwaters of the Shenango River, north of Pittsburgh, Pa., for purposes of flood control, water supply, hydro-electric power development, etc., involving construction of rolled earth dam 2,400 ft. long, maximum height of 50 ft., and steel sheet pile cut-off 40 ft. deep.

Spillways, Gates. Automatic Sector Gates for Coolidge Dam Spillway, P. Baumann. *Eng. News-Rec.*, vol. 109, no. 6, Aug. 11, 1932, pp. 163–166. First installation in United States of automatic control concrete-sector type of spillway gate, developed by Barrages Automatiques Societe Anonyme, Zurich, Switzerland; gates will add 21 per cent to storage capacity of reservoir; results of actual tests of gate action compared to theoretical rise.

Uplift Pressure. Uplift Pressure Measured at Gibson Dam, I. E. Houk. *Eng. News-Rec.*, vol. 109, no. 7, Aug. 18, 1932, pp. 196–198. Observations show absence of uplift at construction joints and only nominal pressures at foundation levels, of arched concrete gravity dam 195 ft. high; uplift measurements at other dams; effects of foundation grouting.

Washington. Cle Elum Earthfill Dam and Outlet Works. *West Construction News and Highways Bldr.*, vol. 7, no. 14, July 25, 1932, pp. 410–419. Description of new U.S. Bureau of Reclamation dam, which will consist of sprinkled and rolled embankment 750 ft. long on crest, a maximum height of 135 ft. above stream-bed, side slopes 3:1 and crest width of 35 ft.; outlet works; construction plant and material supply; river enlargement; gate control shaft; stilling basin.

FLOOD CONTROL

China. Great Flood of Yangtse River, L. Brandt. *Eng. News-Rec.*, vol. 109, no. 5, Aug. 4, 1932, pp. 133–135. Description of the great flood of 1931, which inundated an area of 47,000 sq. miles, and drove more than 10,000,000 people from their homes; range and height of inundation, with calculation of heights, assuming retained river, and some recommendations for immediate palliative measures and ultimate remedy.

FOUNDATIONS

Concrete, Testing. Concrete Piles Tested for Uplift and Bearing. *Eng. News-Rec.*, vol. 109, no. 4, July 28, 1932, p. 104. Results of uplift and bearing tests of piles under new water-supply and purification plant for Fort Wayne, Ind., built on site of municipal dumping ground; tests showed uplift less than settlement for same load.

Construction. Boring Machine Digs Wells for Concrete Piers, H. A. Christie. *Eng. News-Rec.*, vol. 109, no. 4, July 28, 1932, pp. 105 and 106. Description of cylinder piers for Detroit post office; foundations built in wells 118 ft. deep bored in firm clay by special boring machine; boring machine was carried on frame attached to power shovel and operated by motor geared to vertical shaft.

Excavation. Cutting of Sand, J. Rathje. *Engineering*, vol. 134, no. 3472, July 29, 1932, pp. 116–118. Review of paper indexed in Engineering Index 1931; p. 562, from *Forschungsheft*, no. 350, 1931, under the heading of "Excavating Machinery," describing experimental work at Hanover; cutting of sand with horizontal movement of cutter; tests on tractive force showed that deformation of sand proceeded in similar manner to that of metals; cutting resistance on front face of blade; friction on sides of cutter; tests with pointed cutting edges.

Groundwater Control. Ein einfaches graphisches Verfahren zur Berechnung von Grundwasser senkungsanlagen, insbesondere solcher mittels gewebeloser Brunnen, S. Schmah. *Gesundheits-Ingenieur*, vol. 55, no. 17, 18, and 19, Apr. 23, 1932, pp. 193–196, Apr. 30, pp. 208–211, and May 7, pp. 224–226. Development of graphical method for the design of groundwater lowering installations; review of theory of the method and derivation of mathematical formulas, alignment charts, and curves; three illustrative examples.

Mines and Mining, Subsidence. Sicherung von Neubauten gegen Bergschaeden, O. Luetkens. *Gluekauf*, vol. 68, no. 32, pp. 705–711; see also *Iron and Coal Trades Rev.*, vol. 125, no. 3364, p. 277. Protection of new buildings against mining subsidence; method of construction of special foundations, as developed in the erection of the 4,500,000 cu. ft. gas holder at Essen.

Steel, Corrosion. Steel Sheet Piling in Good Condition After 19 years, J. S. Unger. *Steel*, vol. 91, no. 4, July 25, 1932, pp. 26–28. Result of Inspection of U. S. Steel Sheet Piling M-101, 12-in. section weighing 35 lb. per ft., rolled in 1912 by the Carnegie Steel Company, Pittsburgh.

Wooden, Driving. Rotary Drill Bores Holes for Timber Piles in Clay, A. J. Smith. *Eng. News.-Rec.*, vol. 109, no. 5, Aug. 4, 1932, pp. 129 and 130. Equipment similar to modern oil-drilling rig is used on the new municipal auditorium at Long Beach, Calif; piles dropped into holes and driven last few feet; test shows bearing capacity equal to driven piles; provision of drilled holes cut pile driving time in two.

HYDROLOGY, METEROLOGY, AND SEISMOGRAPHY

Floods, United States. Floods of Two Types in Middle Rio Grande Valley, R. G. Hosea. *Eng. News-Rec.*, vol. 109, no. 4, July 28, 1932, pp. 100–102. Streamflow studies show flood peaks dependent upon geographic position rather than area; reservoirs have limited possibilities for flood control; summary of frequency data for spring floods on Rio Grande and tributaries.

MATERIALS TESTING

Concrete, Bond Strength. Comparative Pull-Out Tests, G. P. Manning. *Concrete and Constr. Eng.*, vol. 27, no. 7, July 1932, pp. 381 and 382. Analysis of the tests of six specimens.

Concrete Construction, Costs. Visualizing Concrete Economy in Terms of Strength, I. Lyse. *Eng. News-Rec.*, vol. 109, no. 4, July 28, 1932, pp. 109 and 110. Straight-line relation between strength and cement-water ratio permits a direct measure of cost in terms of strength; cost per unit of strength decreases with the increase in strength.

Strength of Materials. Strength of Materials as Affected by Discontinuities and Surface Conditions, F. C. Lea. *Soc. Glass Technology—Journal*, vol. 16, no. 62, June 1932, pp. 182–205 (discussion), 205–209. Experiments to determine the effect of discontinuities and the surface conditions of metals on the behavior of materials when subjected to cycles of repeated stress; plasticity and discontinuities; experiments on the effect of grooves, etc.; effect of surface conditions produced by heat treatment of steels; effect of corrosion on failure under repeated stresses; impact tests; static and dynamic.

PORTS AND MARITIME STRUCTURES

China. Proposed New Ocean Port for North China. *Engineer*, vol. 154, no. 3995, Aug. 5, 1932, pp. 138 and 139. Creation of a port on the north shore of the Gulf of Pohai is recommended in the report of the Great Northern Port Commission; the project calls for deep-water ocean port, free from ice.

Ireland. Improvements at Waterford Harbor, W. Friel. *Inst. Civ. Engrs. Ireland—Trans.*, vol. 57, 95th session, Nov. 1930 to May 1931, pp. 106–134 (discussion) 134–147, 4 supp. plates. Description of improvements carried out since 1925, including: construction of three steel-decked steel pontoons, erection of custom shed 212 ft. by 32 ft.; building of reinforced concrete wharf 300 ft. by 80 ft.; deepening of navigation channel, etc., cost totaling $80,000.

Railroads. Railroad Service of Port of Stockton, B. C. Allin. *World Ports*, vol. 20, no. 10, Aug. 1932, pp. 815–823. Physical port aspects; individual railroad activities; comprehensive railroad plan; operating agreement; special financial agreement; general advantages; present situation and ultimate development.

Trenton, N.J. Trenton, New Jersey, Port of Entry, B. J. Walsh. *World Ports*, vol. 20, no. 10, Aug. 1932, pp. 805–814. Conversion of river port of Trenton, N.J., into a port terminal; structures and equipment; rail lines and roadways; handling equipment; operation and administration.

ROADS AND STREETS

Asphalt. Asphalt and Its Use for Roads, C. M. Baskin. *Contract Rec.*, vol. 46, no. 27, July 6, 1932, pp. 771–775. Difficulty of old, cylindrical still; steam reducing in shell still; oxidation of asphalt; high pressure and high temperature conversion process. (To be continued.)

Australia. Establishment of Direct Road Communication Between Sydney and Newcastle, T. H. Upton. *Inst. Engrs. Australia—Journal*, vol. 4, no. 6, June 1932, pp. 204–211. Crossing of Hawkesbury River; Kangaroo Point and Mooney Point docks; ferries; service and traffic. (To be continued.)

Bituminous. Asphalts and Road Materials, F. Levy. *Inst. Petroleum Technologists—Journal*, vol. 18, no. 103, May 1932, pp. 391–403. Review of progress for 1930–1931; rock asphalt and natural asphalt; petroleum bitumen; application in road building; emulsions; roofing and paints; specifications and testing. Bibliography.

Canada. Highway Engineering and Road Research in Universities and Laboratories of Canada, P. E. Spielmann. *Roads and Road Construction*, vol. 10, no. 114, June 1, 1932, pp. 168 and 169. Information on principal research, testing, and educational institutions. (To be continued.)

Ceylon. Road Developments in Ceylon, T. B. Fowler. *Roads and Road Construction*, vol. 10, no. 115, July 1, 1932, p. 195. Growth of mechanical transport; primitive equipment; bullock carts and palanquins; main route roads break down; present-day policy.

Chicago, Ill. Highway Policies Defined for Metropolitan Chicago. *Eng. News-Rec.*, vol. 109, no. 7, Aug. 18, 1932, p. 186. Western Society of Engineers formulates thirteen rules of practice for highway development in the Chicago region.

Concrete. Reinforcement in Concrete Bases, J. S. Burch. *Can. Engr.*, vol. 63, no. 2, July 12, 1932, pp. 11–14. Causes of cracking; shrinking during setting period; logical basis of design; practical design considerations; location of reinforcing in slab; methods of installations; essential requirements of method.

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The Society reserves the privilege of rejecting advertisements inconsistent with its ethical procedure.

Construction. Locating and Constructing Trans-Canada Highway in Northern Ontario, S. E. Smith. *Can. Engr.*, vol. 63, no. 7, Aug. 16, 1932, pp. 11–14 and 39–40. Organization and principal features of the work carried out in the Thunder Bay district by the Department of Northern Development; deep ditching through Muskeg Swamp; pole track used on earth cut; improving old roads; labor force numbered 4,500 men distributed in 42 camps.

Germany. Die Kraftwagen-Fernstrasse Koeln—Bonn, H. Ehlgoetz. V.D.I. Zeit, vol. 76, no. 33, Aug. 13, 1932, pp. 794–796. Design and construction of automobile highway from Koeln to Bonn, 20 km. long, 16 m. wide; traffic conditions; details of highway termini at Koeln and Bonn; cost and financing of highway.

Gravel. Versuche mit Oberflaechenbehandlung von Schotterstrassen, O. Ammann and C. V. Gruenewaldt. *Verkehrstechnik*, vol. 20, July 20, 1932, pp. 412–414. Tests of variously treated gravel road pavements, made at circular testing track of the Karlsruhe Institute of Technology; effect of gravel or crushed-rock characteristics on the nature of surface treatment; comparative study of cold and hot tars, asphalt emulsions, etc., as binders.

Highway Administration, Financing. Who Pays for Highways? C. S. Duncan. *Ry. Age*, vol. 93, no. 7, Aug. 13, 1932, pp. 210–216. Actual users of Federal-aid state highways pay only 18 or 20 per cent of construction and maintenance costs; sources of highway dollar; highway policy; additional cost for commercial operation; funds made available for highways and city streets; government vs. private property; relative tax burden.

Intersections. Design of Street and Highway Intersections, L. S. Tuttle and E. H. Holmes. *Pub. Roads*, vol. 13, no. 5, July 1932, pp. 73–88. Recent practice in United States in the design of intersections with and without traffic regulation; increased efficiency for existing intersections through minor changes in physical layout; rotary traffic; existing traffic circles; special designs without rotary principle; existing special designs; progressive development of intersections.

Machinery, India. Markets for Road-Making Machinery in India. *Quarry and Roadmaking*, vol. 37, no. 427, Aug. 1932, pp. 331–333. Present and future road development; cost of maintenance and tarring; particular items of road-making equipment.

Maintenance and Repair. Repairing Utility Cuts and Failures, F. D. Woodruff. *Am. City*, vol. 47, no. 2, Aug. 1932, pp. 53–56. Before Ass'n Highway Officials of North Atlantic States, previously indexed from *Eng. News-Rec.*, April 14, 1932.

Materials, Cotton. Cotton Cloth Used to Waterproof Road Surface, A. Davis. *Eng. News-Rec.*, vol. 109, no. 7, Aug. 18, 1932, p. 194. Demonstration gravel-road surface, with cotton-cloth diaphragm as water seal, constructed by Louisiana Highway Department.

New York City. Non-Skid Pavement of Asphalt Block Has Recessed Transverse Joints. *Construction Methods*, vol. 14, no. 7, July 1932, pp. 28 and 29. Elimination of skidding hazards from heavily traveled section of 6 per cent grade on West 97th St. between Broadway and Riverside Drive, New York City; asphalt blocks are laid against wood spacing strips which form a transverse joint $^1/_2$ in. wide; spacing strips are wood lathe $^3/_8$ in. thick; joint drainage.

Ohio. Street-Widening Experience Counsels Against Haste, E. A. Kemmler. *Eng. News-Rec.*, vol. 109, no. 3, July 21, 1932, pp. 69 and 70. Twelve years of progressive widening by setback lines and direct appropriation in Akron, Ohio, indicates that caution should be exercised in pushing projects unless traffic imperatively demands more lanes.

Preparation Plants. Installs Modern Asphalt Mixing Plant. *Manufacturers Rec.*, vol. 101, no. 28, July 14, 1932, pp. 14 and 15. Outline of plan followed by City of Minneapolis to assure fair bidding in securing plant to meet exacting requirements.

Scotland. County Road Practice in Scotland, G. S. Barry. *Roads and Road Construction*, vol. 10, no. 115, July 1, 1932, pp. 201–203. Contract system; departmental labor system; quarries and production of road material; surface treatment; coatings; grouting; construction for moderately heavy traffic; asphalt for heavy traffic; footpaths; foundation work.

Street Traffic Control. Kreuzungsverkehr und Vorfahrtrecht, F. Albrecht. *Verkehrstechnik*, no. 19, July 5, 1932, pp. 363–365. Layout of crossings with particular regard to factors controlling right-of-way and visibility.

Surface Treatment, Oiling. Road Oils—4, G. M. Mullins. *Nat. Petroleum News*, vol. 24, no. 32, Aug. 10, 1932, pp. 25–26, 28, and 30. Specifications constitute largest problem in connection with road oil; correlation between some laboratory tests and road service is doubtful; costs of road oiling. Conclusion of serial indexed in Engineering Index 1931, p. 1235, from issues of Aug. 12, Sept. 16, and Oct. 7, 1931.

SEWERAGE AND SEWAGE DISPOSAL

Outfall. Sea Outfalls for Sewage. *Engineering*, vol. 134, no. 3471, July 22, 1932, pp. 89 and 90. Review of papers by G. W. Warr and D. Edwards at the Brighton Congress of the Royal Sanitary Institute; results of recent investigations; Southwick outfall; sewerage system of Brighton and Hove.

Pipe, Vitrified Clay. De-Airing System for Sewer Pipe Presses, H. S. Lewis. *Brick and Clay Rec.*, vol. 81, no. 1, July 1932, pp. 14 and 15. Report of study made to improve quality of pipe by use of vacuum device.

Sewage Disposal. Proceedings Sixth Annual Conference, Maryland-Delaware Water and Sewerage Association, Cumberland, Md., April 12 and 13, 1932, 128 pp. Elementary Sewage Disposal, L. V. Carpenter; Admission of Industrial Wastes Into Sewerage Systems, H. M. Freeburn; Symposium—Control of Pollution of Streams, R. C. Beckett, W. L. Stevenson, E. S. Tisdale, A. Wolman; Proper Relation Between Medical Health Officer and Water Department, H. H. Weiss; Influences of Soils, Rocks, and Minerals Upon Potability of Water, J. J. Shank.

Sewers, Concrete. Ausfuehrung eines Kanalbauloses in Muenchen unter Mitverwendung von Vianini-Schleuder-betonrohren, E. Stecher and O. Loesch. *Gesundheits-Ingenieur*, vol. 55, no. 17, Apr. 23, 1932, pp. 198–201. Discussion, by Reichle, of an article on the use of Vianini centrifugal concrete pipe in the construction of a sewer job in Munich, indexed in Engineering Index 1931, p. 1281, from the Aug. 29, 1932 issue.

Sludge. Economic Preparation and Sale of Digested Sludge as Commercial Fertilizer, M. W. Tatlock. *Sewage Works Journal*, vol. 4, no. 3, May 1932, pp. 519–524. Practice of Sewage Treatment Works of Dayton, Ohio; plant of Imhoff sedimentation type is designed for a population of 262,000.

STRUCTURAL ENGINEERING

Arches, Design. Simplification of Arch Design, G. H. Hargreaves. *Concrete and Constr. Eng.*, vol. 27, no. 8, Aug. 1932, pp. 446–455. Variations in arches; dead load on spandrel-filled arches; dead load curves; application of curves. (To be continued.)

Beams, Buckling. Laterally Unsupported Beams, A. N. Proctor. *Structural Engr.*, vol. 10 (New Series), no. 8, Aug. 1932, pp. 340 and 341. Discussion, by A. G. Pugsley, of article previously indexed from issue of July 1932.

Trusses, Welded. New Welded Truss Design Uses Continuous Tension Members. *Eng. News-Rec.*, vol. 109, no. 5, Aug. 4, 1932, p. 136. Description of trusses for new welded buildings to be erected at Derry, Pa., and Mansfield, Ohio—plants of Westinghouse Electric and Manufacturing Co.—in which continuous-strap tension members are used; each strap is welded at its ends to the top chord and bent in such way as to form two diagonals and portion of bottom chord of truss, whose section is increased from ends toward center in such amount that unit-stress remains nearly constant throughout.

TUNNELS

Boston, Mass. Boston Pushes Its Water Program—II, M. R. Budd. *Explosives Engr.*, vol. 10, no. 8, Aug. 1932, pp. 235–241. Blasting operations and concrete lining of Coldbrook, Swift River Tunnel; shaft sinking.

Water Supply, Canada. Completion of Intake and Filtered Water Tunnels. *Contract Rec.*, vol. 46, no. 33, Aug. 17, 1932, pp. 915 and 917–921. Details of the construction of 10-mile concrete-lined tunnels, 8 ft. in diameter, for Toronto's water works extension; map and profile; diagram showing drilling, loading, and shooting arrangements; steel forms used for lining tunnel; guniting.

WATER TREATMENT

Analysis. Fundamental Chemistry of Water Supply: How to Read a Water Analysis, J. H. Coste. *Water and Water Eng.*, vol. 34, no. 406, July 20, 1932, pp. 361 and 362. Rate of fall of water drops through air at atmospheric pressure; composition of rainwater collected in Thames Valley, Bibliography. (To be continued.)

Chicago, Ill. Progress in Filtering Water Supply of Chicago, M. B. Reynolds and A. E. Gorman. *Eng. News-Rec.*, vol. 109, no. 5, Aug. 4, 1932, pp. 137 and 138. Abstract of a paper given before the American Water Works Association, presenting a summary of 63 major testing station experiments and a résumé of engineering studies; conservation of water must precede filters; numerous experimental results incorporated in tentative designs; inshore intakes to replace present cribs; activated carbon in some form to be used.

Chlorination. Chlorination for Special and Emergency Purposes, J. D. MacMahon. *New England Water Works Ass'n—Journal*, vol. 46, no. 2, June 1932, pp. 121–129 (discussion), 129–132. Experience of the Metropolitan District of Boston and small municipal water supplies; disinfection of water mains; control of algae.

Filtration Plants. Design and Operation Data on Large Rapid Sand Filtration Plants in United States and Canada, E. A. Hardin. *Am. Water Works Ass'n—Journal*, vol. 24, no. 8, Aug. 1932, pp. 1190–1207, 1 supp. p. Tabulated data and notes on some forty-eight water filtration plants. Bibliography.

Flocculators. Flocculator Is Important New Device for Water Treatment, A. Anable. *Contract Rec.*, vol. 46, no. 29, July 20, 1932, pp. 815 and 816. Design of Dorrco flocculator and description of eight duplex units installed at Hamilton, Ont.

Idaho. Purification of Snake and Clearwater Rivers, W. P. Hughes. *Am. Water Works Ass'n—Journal*, vol. 24, no. 8, Aug. 1932, pp. 1173–1177. Description of plant at Weiser, Idaho; removal of taste and odors by superchlorination and activated carbon treatment.

Taste and Odor Removal. Some Quantitative Odor Determinations, C. H. Spaulding. *Am. Water Works Ass'n—Journal*, vol. 24, no. 8, Aug. 1932, pp. 1111–1118. Review of methods proposed for the measurement of the intensity of obnoxious smells; dilution method of measurement; experience of water-softening plant of Bloomington and Springfield, Ill.

Water Supply of Toledo, R. W. Furman. *Water Works Eng.*, vol. 85, no. 14, July 13, 1932, pp. 846–850. Experience of the Water Department of Toledo, Ohio, in remedying taste and odor from the Maumee River water; tastes from algae and microorganisms; decomposition of algae; types of floc; remedy for secondary floc conditions; use of activated carbon; hourly loss of head while using pulverized activated carbon in filter; treatment for great quantities of algae; tests conducted on individual filter; taste and odor removal through adsorber.

Washington. Chlorination of Water Supplies in State of Washington, H. W. Nightingale. *Am. Water Works Ass'n—Journal*, vol. 24, no. 8, Aug. 1932, pp. 1178–1182. Methods of safeguarding supplies; waters not requiring preliminary treatment; waters requiring preliminary treatment; extent of chlorination.

WATER WORKS ENGINEERING

Accounting. Accounting for Water Districts, J. W. Graham. *New England Water Works Ass'n—Journal*, vol. 46, no. 2, June 1932, pp. 196–200. Fixed capital account; depreciation reverse account; equalization and apportionment.

Cuba. Water Works in Cuba, J. F. Pierce. *New England Water Works Ass'n—Journal*, vol. 46, no. 2, June 1932, pp. 184–195. Review of geography of island; topography; rivers; geology; groundwater; work of U.S. army; inadequacy of supplies; rainfall.

Dutch Indies. Water Systems of Java, W. Buchler. *Water Works Eng.*, vol. 85, no. 15, July 27, 1932, pp. 898–900. Notes on combined river and mountain supply for Soerabaya; purification of river water; coin meters substituted for water sellers; public bathing houses for natives; water diseases.

Great Britain. Water Supply of Bournemouth, P. G. G. Moon. *Water and Water Eng.*, vol. 34, no. 406, July 20, 1932, pp. 346–353. History and description of the water works, including pumping stations, water softening, and filtration plant, of a fashionable watering place, having a population of 116,000.

Management. Obligations of Water Authorities, W. Cash. *Water and Water Eng.*, vol. 34, no. 406, July 20, 1932, pp. 343–345. Abstract of presidential address opening the 1932 annual meeting of the British Water Works Association; purification processes; water charges; working costs; road improvements and water mains; service pipes maintenance.

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